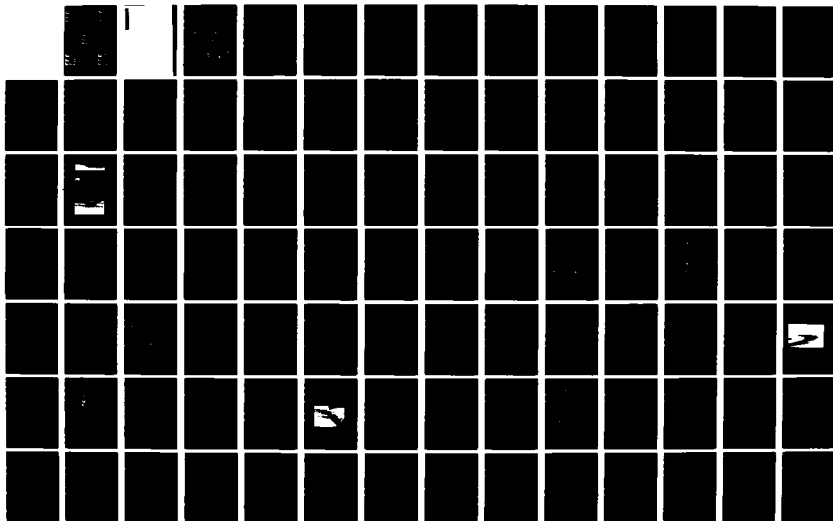


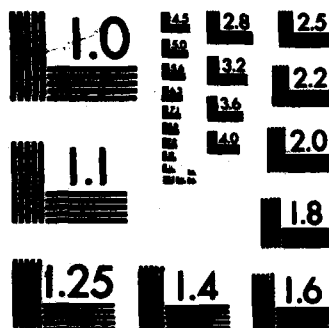
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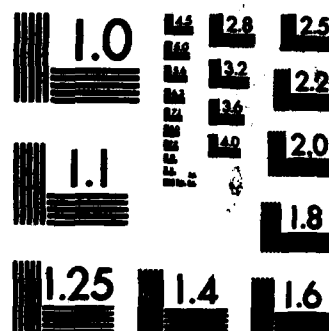
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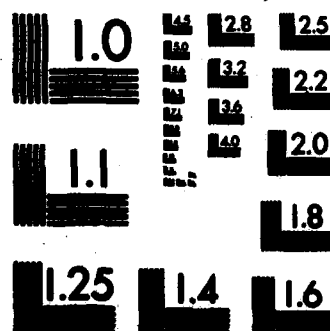




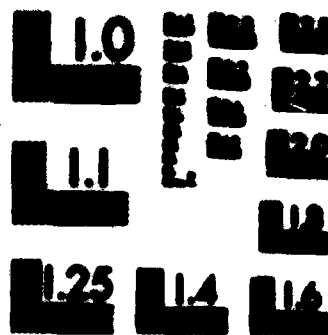
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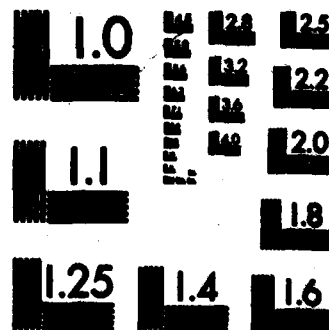
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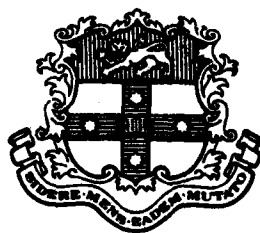
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July, 1982

BORES AND SWASH ON NATURAL BEACHES

MARK BRADSHAW



COASTAL STUDIES UNIT
DEPARTMENT OF GEOGRAPHY
THE UNIVERSITY OF SYDNEY
SYDNEY, N.S.W. 2006

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ABSTRACT

A field study has been carried out to investigate the behaviour of bores in the inner surf zone of natural sandy beaches and to examine a theory which links bores to swash through a process of bore collapse at the shoreline. The theory models subsequent swash as a lens of water moving up the beach slope under the decelerating influence of gravity only. Cine-photography was used to collect all data. Bores were filmed on a flat profile in shallow water ($<0.5\text{m}$) which was either at rest or flowing seaward with velocities up to approximately 1 m/sec . In all cases it was found that the theoretical velocity of the bore front (calculated on the basis of water depths on either side of the bore and taking into account the velocity of the underlying water), correlated closely with observed velocity. Film records of bore collapse on a steep and a flat beach and of subsequent swash flows indicate that the bore disappears at the shoreline in both cases. This occurs as a gradual flattening of the steep bore face over distances ranging from 1.5m (small bore-flat beach) to 5m (large bore-steep beach) and is associated with acceleration of the leading edge of water. Swash velocities at the base of the steep beach were found to be high compared to those hitherto reported. However, the initial velocity of swash issuing from the small bore (flat beach) was found to be greater, relative to bore height, than those observed on the steep beach. Swash



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flows on both beaches were found to fall short of theoretical predictions and this indicates that the effects of friction and percolation of water into the beach need to be incorporated into the model.

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The theory of bore propagation over a sloping bottom is used to simulate the behaviour of multiple bores in a surf zone, and this exercise shows the degree to which bore concatenation is theoretically possible under different slope and wave energy conditions. Results can be used to partially explain the low frequencies that characterise swash on flat beaches.

ACKNOWLEDGEMENTS

The collection of data in the coastal zone usually requires the involvement of many people and the project reported here is no exception. The task of providing field assistance has fallen mainly to my colleagues in the Coastal Studies Unit: Don Wright (who supervised the project), Andy Short, Mal Green, Felicity Coffey and Graham Lloyd. I thank you all for your assistance and support.

In particular I extend warm thanks to Peter Cowell who provided hours of stimulating discussion and a great deal of inspiration, and to Peter Nielsen who helped clarify many theoretical aspects of the study. Discussions with John Chappell were also of great help.

Additional field assistance was provided by Paul Cameron, Paula Douglas, Heidi Erber, Jenny Lanyon, Anthony Learmonth, Ann-Marie Mitford, Cherie Poulton, Karen Riley and Neil Trenaman. John Farrow and Lori Mooren provided help on the beach in the final stages of the study.

The following people contributed to the production of the report: John Roberts and Johanna de Roder (illustrations), Janette Martin, Suzanne Gronow and Wendy Nickalls (typing and printing), Mal Green and Claire Fisquet (proof reading and final collation). Thank you all for your help.

Much of the data reported herein was collected while I was working under the direction of Assoc. Prof. L.D.

Wright, on projects funded by the Australian Research Grants Committee and the U.S. Office of Naval Research, Coastal Studies Program, Task NR 388-157, Grant N-00014-80-G-0001. The support of these agencies is gratefully acknowledged. Financial assistance was also provided by the University of Sydney through a University Research Grant.

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LIST OF SYMBOLS

A	a constant.
C	wave celerity.
C_h	Chezy coefficient.
H	wave height.
H_b	breaker height.
K	volume of water in bore per unit width.
L	wave length.
M	bore strength (W/\sqrt{gh}).
R	maximum vertical run-up height above still water level.
T	incident wave period.
U_s	velocity of leading edge of swash.
U_o	horizontal water velocity at bore collapse.
W	bore front velocity.
a	incident wave amplitude.
d	water depth at break point.
f	a friction coefficient.
g	acceleration due to gravity.
h	water depth behind bore face (high side).
h_o	water depth in front of bore face (low side).
h_s	swash depth.
h'	water depth a short distance behind bore face.
k	a constant.
l_s	maximum horizontal swash excursion.
t	time.
u	horizontal water particle velocity behind bore face.
u'	water particle velocity a short distance behind bore front.

u_d	velocity of water in front of bore face.
x	horizontal distance.
x_s	instantaneous shoreline position.
α	slope in degrees.
γ	bore height-to-depth ratio (γ/h_0).
ξ	dimensionless surf scaling parameter.
η	height of bore face.
ω	incident wave radial frequency.
obs	observed.
est	estimated from theory.

CHAPTER 1

INTRODUCTION.

1.1 SCOPE OF THE STUDY

This report is concerned with the behaviour of water in the inner surf zone and swash zone of sandy beaches and with the relationship between surf and swash processes on the one hand, and inshore slope on the other. The region constitutes the interface between the subaerial beach and the sea and although it is only a small part of the total beach system, it is nevertheless significant both in terms of its variability and, more importantly, in terms of its role in shaping the beach face and, indirectly, the zones on either side.

The theory of bore propagation over a sloping bottom and the transformation of bores to swash at the shoreline is discussed and aspects of the theory are compared to observations made on steep and flat beaches.

In addition, a model based on bore theory is used to examine the extent to which bore concatenation may occur across a surf zone. The model is run for different beach slopes and wave energy conditions and the predictions are compared to field observations.

1.2 AN OVERVIEW OF THE LITERATURE

Since the report by V. Cornish in 1898 on observations of swash-backwash flows on shingle beaches and their relationship to sediment movement, surf and swash zone processes have been studied from many viewpoints.

A majority of early workers concentrated on the broad themes established by Cornish, attempting to discover the links between swash and surf characteristics, sediment sorting and transport mechanisms and patterns of sediment distribution. Typical of these are studies by Evans (1939), Bascom (1951), Miller and Zeigler (1958), Ingle (1966), Friedman (1967) and more recently, James and Brenninkmeyer (1977). Allied works like Strahler (1966) link swash zone processes to changes in beach face morphology over short time periods (ie. a tidal cycle). Small scale morphologic features, particularly beach cusps have been well documented. Sallenger (1979) contains a summary of relevant papers.

The importance of the beach water table as a factor in both short and long term changes in beach face morphology has been stressed by Emery and Foster (1948), Grant (1948), Duncan (1964), Bradshaw (1974), Chappell et al. (1979) and Lanyon (1979). The dynamics of the water table have been studied by Harrison et al. (1971), Waddell (1973) and Lewandowski and Zeidler (1978).

The hydrodynamics of swash and backwash flows have been treated theoretically for the cases of both breaking and non-breaking waves. Initially, non-breaking surge on steep slopes was assumed (Miche, 1944; Lewy, 1946; Isaacson, 1950; Carrier and Greenspan; 1958). Later,

theories for surf and run-up on a breaker dominated shoreline were developed by Keller et al. (1960), Ho et al., (1963), Freeman and LeMehaute (1964) and Amein (1966). These focus on the behaviour of bores propagating across a surf zone and their conversion to run-up at the shoreline. More recently, Guza and Bowen (1976) and Guza and Thornton (1982) have presented work based on the hypothesis that the surf zone contains partially reflected standing wave components which cause run-up at the shoreline and dissipative breaking components which have no associated run-up. Using laboratory and field data they test Miche's (1944) hypothesis that the standing wave amplitude at the shoreline, with incident waves breaking, is equal to the maximum that can occur without breaking.

Early empirical work on run-up was conducted in wave tanks and was concerned with finding predictive formulae for maximum run-up height, given input wave height and period and assuming a regular wave train. Examples are Grantham (1953), Hall and Watts (1953), Kaplan (1955), Saville (1958) and Hunt (1959).

Hydrodynamic observations from natural beaches have been reported by Emery and Gale (1951), Waddell (1973), Sonu et al. (1974), and Huntley and Bowen (1975). Discussion in these is mainly centered on the frequency of run-up on beaches although Waddell (1973) and Huntley and Bowen (1975) also mention swash-backwash flow velocities. Internal flow characteristics have been further examined by Kemp and Plinston (1974), Kemp (1975) and Roos and Battjes (1976) in laboratory experiments while Kirk (1975) reports on time averaged velocities on a steep gravel beach.

The purpose of the foregoing precis is not to present a comprehensive review of a large body of literature but rather to demonstrate the scale and diversity of the approaches taken. More complete summaries of the material are contained in LeMehaute et al. (1968), Webber and Bullock (1971), Meyer and Taylor (1972), and the report of the Technical Advisory Committee on Protection Against Inundation (1974).

1.3 AIMS AND APPROACH OF THE STUDY

There are two major deficiencies in the area of run-up research as applied to sandy beaches.

The first concerns the lack of a well tested body of theory to describe the motion of water in the vicinity of the beach face on a wave-by-wave basis. Much analytical work has been carried out on the behaviour of non-breaking waves near the shoreline but these are of limited relevance to the study of natural beaches. Of far greater potential relevance is a theory based on the propagation of bores across a surf zone and their transformation to run-up at the shoreline which has been discussed in detail by Keller et.al (1960), Ho et al. (1963), Freeman and LeMehaute (1964) and others. However, the theory has not been widely embraced, and partial evidence of this is the almost total lack of verification using laboratory and field data.

The second deficiency is inherent in most swash zone field studies and has to do with the disregard that these show for the interrelationship between inshore topography and hydrodynamic processes. The idea that the morphologic

and hydrodynamic characteristics exhibited by a beach at any given time are the result of mutual interaction and coadjustment between processes and beach form has been discussed in general terms by Wright and Thom (1977) and, with reference to specific case studies, by many including Chappell and Wright (1978), Chappell and Eliot (1979), Short (1979) and Wright et al. (1979). However, while several works deal with the hydrodynamic and sedimentological characteristics of the inner surf zone and swash zone, few construct the important link between these and beach morphology in general and, in particular, beach slope.

As part of this study, data has been collected from both steep and flat beaches on: (i) the behaviour of bores as they travel through shallow water towards the 'dry' beach face where they turn into swash, and (ii) the frequency of water motions in the shallow water of the inner surf zone and in the swash zone.

The aims of the study are twofold. The first is to assess the applicability of bore theory for the study of surf and run-up processes on both steep and flat beaches. This is done using the data collected on the behaviour of individual waves as they approach and cross the shoreline. The second is to model the behaviour of successive bores in a surf zone using bore theory and then, to compare the results to the observed frequency characteristics of steep and flat beaches. Achievement of these aims will hopefully go some way towards removing the deficiencies noted above.

Chapter 2 begins with a consideration of theories of surf. The theory of bore propagation through the surf zone

and its conversion to a 'rarefaction wave' (the swash) at the shoreline (Keller et.al, 1960; Ho et al., 1963; Freeman and LeMehaute, 1964) is argued to be highly relevant to the study of run-up on the basis that bores dominate the inner surf zones of flat beaches and are seen to immediately precede the swash phase. Details of the theory are outlined and the lack of field verification is noted.

In Chapter 3, the results of observations of water motion on and slightly seaward of the beach face on a flat profile are presented. Film records provide data to test bore theory's ability to accurately predict bore velocity in shallow water. Observations of bore collapse and run-up are also described.

The applicability of the theory to water motion on steep profiles is examined in Chapter 4 by describing the behaviour of different types of breakers near the shoreline. Some data on swash velocities and swash excursion widths are also presented.

In Chapter 5 successive bores are modelled as they progress across a surf zone. The model is based on the theory discussed and tested in Chapters 2 and 3. It predicts bore velocity and position, given a uniform beach slope and primary breaker characteristics (both of which can be varied) and thus indicates the possible extent of bore-bore overrun for a given breaker-beach slope combination. These predictions are discussed in the light of observations of water motion frequencies from natural beaches.

A summary and conclusions are presented in Chapter 6.

1.4 SOME DEFINITIONS

This report examines beaches which have contrasting morphodynamic characteristics and which are very different in the visual sense. It is therefore useful to define the regions of interest particularly for the two extremes of a steep and a flat beach.

The concept of a 'shoreline' is central to all subsequent discussions and refers to the intersection of still water level with the beach slope. On an idealised profile, the swash zone or 'dry beach' lies landward of the shoreline and is inundated periodically by wave uprush. Below the shoreline lies a region dominated by breaking waves and bores which is always covered by water. In reality, the regions are not so easy to define because boundaries are often migratory and/or indistinct.

Steep beaches are the easiest to deal with, having two readily identifiable features: a high, prominent berm crest and a step at the base of the slope. Under most conditions, waves will break near the step, propelling water up the beach face. The surf zone will be very narrow (extending only a few metres beyond the step) and the swash zone will occupy the area between step and berm (figure 1.1a). The whole system will be fixed in space, moving only in response to tides.

On the other hand, flat beaches display an easily recognisable surf zone but a poorly defined swash zone. After breaking, waves must travel some distance before reaching the 'dry beach'. On the way there is significant wave-wave interaction and this, combined with an oscillating inshore water level (due to low frequency

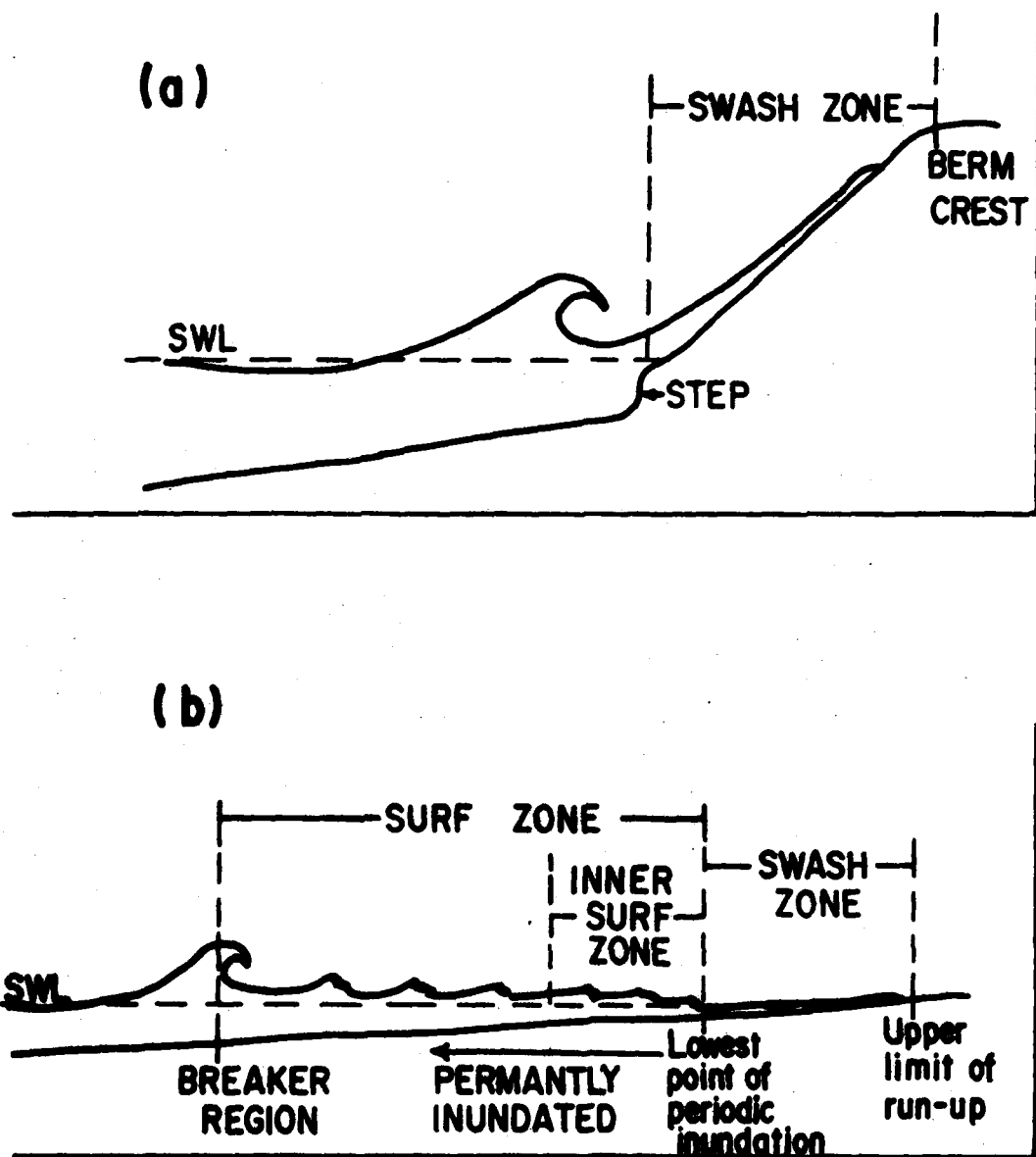


FIGURE 1.1 Definition sketches showing surf and swash zones on steep and flat beaches.

fluctuations in set-up) results in a shoreline which moves considerably over a wide area of sand, usually with a period significantly longer than that of the incident waves. Run-up under these circumstances can be treated as a movement of the shoreline due to a combination of incident waves and the longer term surf zone water level movements (Van Dorn, 1976). However, even on the flattest of beaches it is usually possible to discern a narrow region below which the beach is never exposed and this is often marked by the regular occurrence of a sediment laden hydraulic jump which results from the interaction of a strong backwash with an oncoming bore. For the purposes of this work it will constitute the lower limit of the swash zone (figure 1.1 b).

Observations of wave activity in the 'inner surf zone' are presented in Chapters 3 and 5 and this refers to the region extending seaward from the shoreline to a depth of approximately half a metre.

CHAPTER 2

BORES AND RUN-UP: THEORETICAL CONSIDERATIONS.

Over the past fifty years a large amount of literature has addressed the problem of run-up by attempting to find analytical solutions for the phenomenon. Early efforts were underpinned by the well defined aim of predicting run-up excursions from a knowledge of beach slope and deep water wave characteristics. Now, as Meyer and Taylor (1972) point out, the more sober aim is to "understand the nature of water motion in the immediate vicinity of the shoreline". This requires an examination of not only run-up per se but also of the processes leading up to it and to this end several theories have been developed (see reviews by LeMehaute et al., 1968; Meyer and Taylor, 1972; Webber and Bullock, 1971).

In this chapter I review some of the theories which consider breaking waves. Studies of run-up resulting from non-breaking waves are a special case, of limited relevance to our understanding of natural beach processes. A summary of theories dealing with the non-breaking case is available in a report by the Technical Advisory Committee on Protection Against Inundation (1974). In particular, I concentrate on the theory of bores on a sloping beach (Keller et al., 1960) prior to presenting salient field data in Chapters 3 and 4.

2.1 BORES AND NON-SATURATED BREAKERS

Two types of waves dominate the surf zones of flat beaches; non-saturated breakers (LeMehaute, 1962) and bores (Keller et al., 1960).

The model non-saturated breakers considered by LeMehaute (1962) are symmetrical about the crest with breaking taking the form of a gentle spilling of water down the face of the wave (Figures 2.1a and 2.2). The wave begins to break when the wave height to water depth ratio exceeds the maximum allowable for a solitary wave. The limiting value for this ratio has been theoretically derived by many (reviewed by Galvin, 1972) with 0.78 (McCowen, 1894) being the most often quoted. As the wave moves into shallow water, energy flux will be reduced by spilling and by bottom friction and if this happens at a rate sufficient to maintain a wave height below the allowable maximum the wave will progress shorewards without forming a steep, unstable front. LeMehaute (1962) calls such a wave a non-saturated breaker. Theoretically, a non-saturated breaker which reaches the shoreline will have dissipated all incident wave energy and will produce no run-up at this frequency. There will however be an elevation of inshore water level due to the mass transport and momentum flux of the waves (LeMehaute et al., 1968). Non-saturated breaker theory is relevant to the study of beach run-up because for some combinations of incident wave and inshore characteristics it predicts no run-up.

LeMehaute (1962) notes that bores result when energy flux can no longer be dissipated by gentle spilling and bottom friction. This leads to a loss of symmetry and a steep wave face moving into water which is essentially

undisturbed. Unlike the approach of a non-saturated breaker which causes a rise in water level prior to the arrival of the wave crest, a bore transmits no forewarning of its approach. Water will tumble down the face creating a great deal of turbulence but this is peripheral to the definition (see section 2.2). Bores on a natural beach are shown in figure 2.3. Theoretically, horizontal velocities behind the face of a bore are uniform through the water column (figure 2.1b). Peregrine (1966) classifies bores according to their height-to-depth ratio (γ). For $\gamma < 0.28$ the bore is undular and consists of a series of undulations radiating behind a leading wave. Partially developed bores occupy the range $0.28 < \gamma < 0.75$; trailing undulations still exist but the leading wave is breaking. For $\gamma > 0.75$ the bore is fully developed and is of the type shown in figure 2.1b.

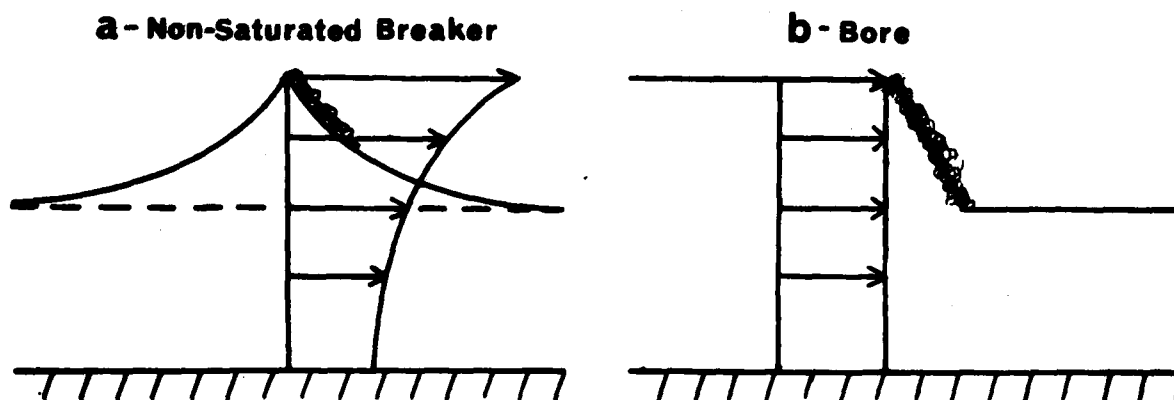


FIGURE 2.1 Theoretical shapes of a non-saturated breaker and bore showing current profiles. (after LeMehaute, 1962)



FIGURE 2.2 **Non-saturated breakers on a natural beach.**
(Photograph by P. Cowell)



FIGURE 2.3 **Bores on a flat beach.**

The question now arises: to what extent, or in what proportions, are spilling breakers and bores of different kinds represented in natural surf zones. To my knowledge, no systematic field study addresses this problem. On the contrary, most field workers fail to draw a clear distinction between these two fundamentally different forms of surf zone water motion. However, based on personal observations of a large number of flat beaches, I would suggest that bores dominate the shallow inner surf zone immediately seaward of the 'dry' beach face (see definitions in section 1.4). Run-up on flat beaches is always seen to issue from a fully developed bore.

My general contention is that bores play a major part in determining modes of water motion on a large number of beaches and thus, are important to our understanding of surf zone processes and run-up dynamics. The remainder of this chapter is devoted to a discussion of the theoretical aspects of bores and run-up.

2.2 BORE THEORY

The motion of a bore over a sloping beach was first discussed by Keller et al. (1960) and later taken up by Ho and Meyer (1962), Shen and Meyer (1963), Ho et al. (1963), Freeman and LeMehaute (1964) and Amein (1966). These follow from Stoker's (1957) analysis of bore formation and development and like Stoker, start with a consideration of the shallow water long wave equations.

Using the definition diagram (figure 2.4), let x be horizontal distance, t be time, g be gravity, $h(x,t)$ the local water depth (ie under the wave), $h_0(x)$ the still

water depth, and u the water velocity component in a shoreward direction. Then the first order non-linear wave equations for 2-dimensional water motion in shallow water are:

Continuity

$$\frac{\partial h}{\partial t} + \frac{\partial(hu)}{\partial x} = 0 \quad \dots (2.1)$$

Motion

$$\frac{\partial u}{\partial t} + u \frac{\partial u}{\partial x} + \frac{g \partial(h-h_0)}{\partial x} = 0 \quad \dots (2.2)$$

(Stoker, 1957; p291)

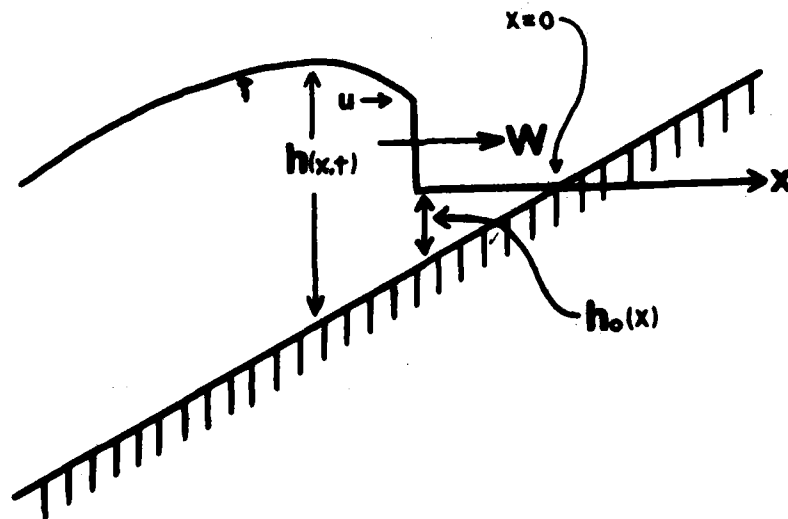


FIGURE 2.4 Bore theory definition diagram.

- h - water depth under wave (behind bore face)
- h_0 - depth of water immediately in front of bore face.
- W - velocity of bore front.
- u - velocity of water particles behind bore front.

Meyer and Taylor (1972) note that these equations are not applicable when waves steepen and break near the shoreline. The connection between them and steep surf is made by first noting that the steep wave front occupies a relatively short horizontal distance and then by setting up a model which treats this region as a local discontinuity on either side of which the shallow water equations apply. Thus, regardless of what happens in the region of the wave front, mass and momentum are conserved over all (although Meyer, 1970, points out that energy will not be conserved and that dissipation will increase as the height of the discontinuity increases; the physical manifestation is the turbulent face).

Meyer and Taylor (1972) call this discontinuity a bore - a region in which the details of water motion are irrelevant. The model deals only with questions of bore position, velocity and strength (change in total water depth across the bore) regardless of whether or not the steepening of the wave front leads to breaking. Further, bore width is not considered; the region may be as narrow as the vertical face of a breaker or, in the case of weak bores, many times wider than water depth.

The classical fully developed bore depicted in figure 2.1b is a special but nevertheless important case. We can readily observe this type of bore on a flat beach. However it is not the only case since it will be shown that it is also plausible to apply the bore model to the region of breaking on a steep beach.

The basic bore velocity equations are given by Stoker (1957) and have been restated by Keller et al. (1960)

and Freeman and LeMehaute (1964). The latter give:

$$W = \sqrt{\frac{gh(h + h_0)}{2h_0}} + u_d \quad \dots (2.3)$$

and
$$u = W \left[\frac{h - h_0}{h} \right] \quad \dots (2.4)$$

(pp195-196)

where W is the velocity of the bore,

u is the water particle velocity immediately
behind the bore front,

and u_d is the velocity of the underlying water
(ie. on the low side of the bore).

Note: These equations are derived from the
Freeman-LeMehaute dimensionless forms by substituting
dimensional terms (ibid; pp214-216).

It is important to note that, unlike solitary waves,
bore velocity depends not on wave height but on the height
of the bore face, $\eta = h - h_0$, relative to the depth of
water into which it is travelling (h_0). Both the
height-to-depth ratio ($\gamma = \eta/h_0$) and the bore strength, M ,
where,

$$M = W/\sqrt{gh} \quad \dots (2.5)$$

(Keller et al., 1960; p304)

have been shown theoretically to increase shorewards as h_0
decreases (Keller et al. 1960). Unlike solitary waves, the
height-to-depth ratio of a bore is not constant as the
bore moves shorewards across the surf zone. Near the
shoreline, the ratio can be many times greater than
unity.

2.3 BORES AND RUN-UP

The theoretical behaviour of a bore moving into water of decreasing depth has been reviewed by Stoker (1957) at some length in the context of dam breaks. The same problem, as applied to the movement of a bore towards the shoreline of a sloping beach, has been investigated by Keller et al. (1960), Ho et al. (1963), and Freeman and LeMehaute (1964). The paper by Ho et al. (1963) summarises the highly technical arguments presented by Ho and Meyer (1962) and Shen and Meyer (1963).

According to Stoker (1957), a dam bursting into water of depth h_0 will generate a shock wave (a bore) which will travel down-stream with a velocity, W , given by equation 2.3. If h_0 is decreasing down-stream and if the bore height remains constant, then W will tend to infinity as $h_0 \rightarrow 0$. However, Stoker notes that as the water in front of the bore decreases in depth, the bore height also tends to zero and the bore accelerates. In the context of beaches, Keller et al. (1960) arrive at the same solution. They demonstrate that $\eta \rightarrow 0$ as $h_0 \rightarrow 0$ and show numerically that both W and u increase shoreward.

That the bore collapses at the shoreline has also been arrived at independently by Ho et al. (1963) and Freeman and LeMehaute (1964). The latter stress that the disappearance of the bore marks the transition from a shock wave to a 'rarefaction' or 'depression' wave (the run-up), and note the similarity to the case of a dam break onto a dry bed. Stoker's (1957) solution to this problem is not a bore, but a depression wave with a parabolic shaped front and an acute leading edge. Freeman and LeMehaute show that friction causes the leading edge

to be cut short, thus giving the run-up a bore-like appearance (figure 2.5).

The bore collapse phase is marked by a rapid conversion of potential to kinetic energy (Meyer, 1970). Both W and u increase to a terminal velocity U_0 which is the horizontal velocity attained by the water when h_0 goes to zero. This is given by:

$$U_0 = u' + 2\sqrt{gh'} \quad \dots (2.6)$$

(Freeman & LeMehaute, 1964; p198)

(Amein, 1966; p407)

where u' and h' are water speed and water depth a short distance behind the bore front.

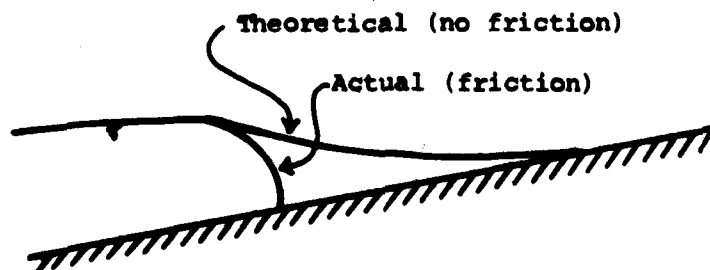


FIGURE 2.5 Theoretical and actual shape of the swash front on a dry bed (after Freeman & LeMehaute, 1964)

Ho et al. (1963) assume that the water body moving on the beach face after bore collapse can be divided into small fluid elements bounded by the sand and the free surface and separated from each other by vertical division planes (figure 2.6). Further, they assume that each element contains the same mass of water at all times. If friction is ignored, the motion of each element will depend only on gravity and the pressure exerted by adjacent elements and the leading element in the run-up sheet will move solely under the influence of gravity. Thus, run-up on the beach is made analogous to the motion of a particle projected up an inclined plane with some starting velocity, U_0 .

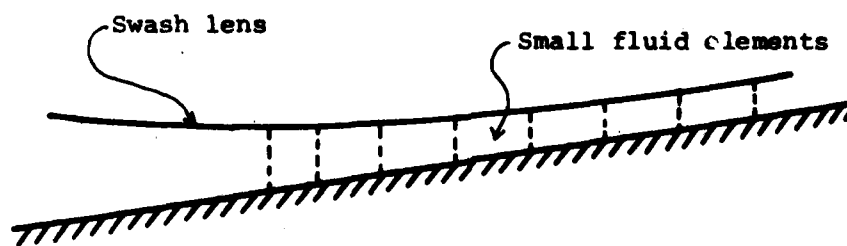


FIGURE 2.6 Idealized model of run-up.
(after Ho et al., 1963)

Accordingly, a set of equations can be written to describe the motion of the leading edge of the swash (Ho et al., 1963; pp226-227). The velocity of the leading edge (U_s) at any time t will be given by:

$$U_s = U_0 - gt \tan \alpha \quad \dots (2.7)$$

where $-g(\tan \alpha)$ is the deceleration due to gravity on a gentle beach slope, inclined at an angle of α degrees to the horizontal.

The position of the instantaneous shoreline (x_s) at time t will be:

$$x_s = \cos \alpha (U_0 t - \frac{1}{2} g t^2 \tan \alpha) \quad \dots (2.8)$$

(The $\cos \alpha$ term can be assumed equal to 1 for practical purposes ie. slopes $< 10^\circ$).

The swash will reach its maximum excursion width when

$$\frac{dx_s}{dt} = U_s = 0 \quad \dots (2.9)$$

that is when

$$t = \frac{U_0}{g \tan \alpha} \quad \dots (2.10)$$

The value of x_s when $dx_s/dt=0$ will give the maximum horizontal swash excursion, f_s , and the corresponding maximum vertical run-up height above still water, R :

$$f_s = \frac{U_0^2}{2g \tan \alpha} \quad \dots (2.11)$$

and
$$R = \frac{U_0^2}{2g} \quad \dots (2.12)$$

The question of bore shape and its effect on run-up is raised by Amein (1966) who distinguishes between major and minor bores. The classification is unrelated to that of Peregrine's (1966) and has been devised to indicate the height of the bore face relative to the body of water immediately behind it (figure 2.7). Using wave height at the toe of the beach slope as a reference, a bore is classified as major if the ratio of bore height to reference wave height is greater than 0.5. This type of bore is favoured by flatter slopes and short period waves. It is classified as minor if this ratio is less than 0.5. Using the method of characteristics to determine the maximum run-up resulting from the two different bore types, Amein finds that the height of run-up issuing from a minor bore will be determined not by the leading bore elements but by the wave elements in the vicinity of the wave crest. This is because the water in the wave crest will tend to catch and overrun any initial run-up from the region of the bore face.

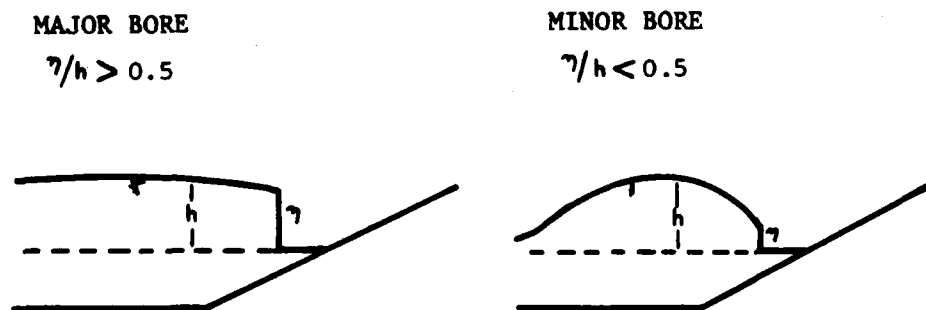


FIGURE 2.7 Amein's (1966) bore classification scheme.

2.4 THE EFFECT OF FRICTION AND TURBULENCE

The analysis above ignores friction and turbulent energy dissipation (as well as other factors such as water seepage into the beach) so it gives only maximum values for run-up height, width and duration. Meyer (1970) suggests that loss of energy due to turbulence will be most important during the bore collapse phase when there is a rapid conversion of potential to kinetic energy, and that this will have a major effect on run-up. Freeman and LeMehaute (1964) note that one effect of bed friction on run-up will be to cut short the leading edge so that it takes on the physical appearance of a bore (figure 2.5).

On a quantitative level, little has been advanced to deal with the problem. Freeman and LeMehaute (1964) propose that run-up height may be reduced by a factor of

$$\frac{(1 + A)(1 + 2A)}{1 + f/A^2 \tan \alpha} \quad \dots (2.13)$$

Here, A is a constant in the equation $U = AC$ where $C = \sqrt{gh_s}$ (h_s = swash depth) and f is equal to g/C_h^2 where C_h is a Chezy coefficient. However, empirical testing and calibration of eq. 2.13 has been minimal (this is further discussed in Chapter 3).

2.5 IRREGULAR WAVES

The theoretical discussion so far has been based on the assumption that no interaction takes place between bores and backwash and that bores do not overtake each other in the surf zone. When this assumption is discarded, as it must be in the case of natural beaches,

the problem of bore development and run-up becomes more complex. Natural waves are not only irregular but are rarely of a period sufficiently long to enable backwash to be completed before the onset of the next wave.

The report of the Technical Advisory Committee on Protection Against Inundation (1974) notes that "...no theories are known concerning the run-up of irregular waves working on the basis of the laws of mechanics and probability theory". However, Meyer and Taylor (1972) maintain that no new fluid mechanism is introduced by the interaction of bores and backwash and Peregrine (1974) has made a start at dealing with the problem by describing in a 'mathematically based qualitative way' what happens when a variety of interactions occur in the surf zone. Peregrine's analysis is based on the method of characteristics and describes the type of waves formed when bores meet or overrun in the inner surf zone.

CHAPTER 3

OBSERVATIONS OF BORES AND RUN-UP ON FLAT BEACHES.

In Chapter 2 some theories based on the non-linear long wave equations were examined in relation to beach run-up processes. Bore theory in particular received close attention because of the fact that fully developed bores can readily be observed to dominate the inner surf zones of flat beaches.

While bore theory and its relationship to the run-up process receives considerable treatment in the literature (at least for the simple case of regular waves on a frictionless beach), reports of laboratory and field tests are by no means plentiful. At this stage, the extent to which bore theory adequately accounts for the behaviour of bore-like waves in natural surf zones is largely unknown.

For this reason, I have conducted experiments on steep and flat beaches which provide new data for the evaluation of some aspects of the ideas discussed in Chapter 2. In this chapter I present those relating to flat beaches, prefaced by a discussion of experimental work to date.

3.1 PREVIOUS WORK

The essence of the theory which attempts to model the behaviour of a bore as it traverses a surf zone and runs up on a beach are encapsulated by the following points:

- (i) the velocity of a bore can be computed from a knowledge of its height and the depth and velocity of the water into which it is travelling (eq 2.3). As the bore moves into water of decreasing depth, both its velocity and strength increase. The limiting velocity, U_0 , is attained the instant the dry beach is reached.
- (ii) at this point the bore collapses and turns into a rarefaction wave. Here it takes the form of a thin sheet of water travelling up the slope with initial velocity U_0 , and under the decelerating influence of gravity. Ignoring friction, the model proposed for the movement of its leading edge uses the simple equations of motion of a particle projected up an inclined plane (eq 2.7 - 2.12).

Miller (1968) provides the only major data set which bears directly on the above. His wave tank experiments, using artificial slopes ranging from 2 to 15 degrees and both fully and partially developed bores, address three questions:

- (i) how does the celerity of the bore front change as it progresses over the slope,
- (ii) how does the shape of the bore front change, particularly when it encounters the shoreline, and
- (iii) is run-up height correctly predicted by equation 2.12, regardless of the slope angle (note that the equation is independent of slope)?

Some of Miller's results are summarised in figures 3.1 to 3.4. In general, they show a reasonable qualitative

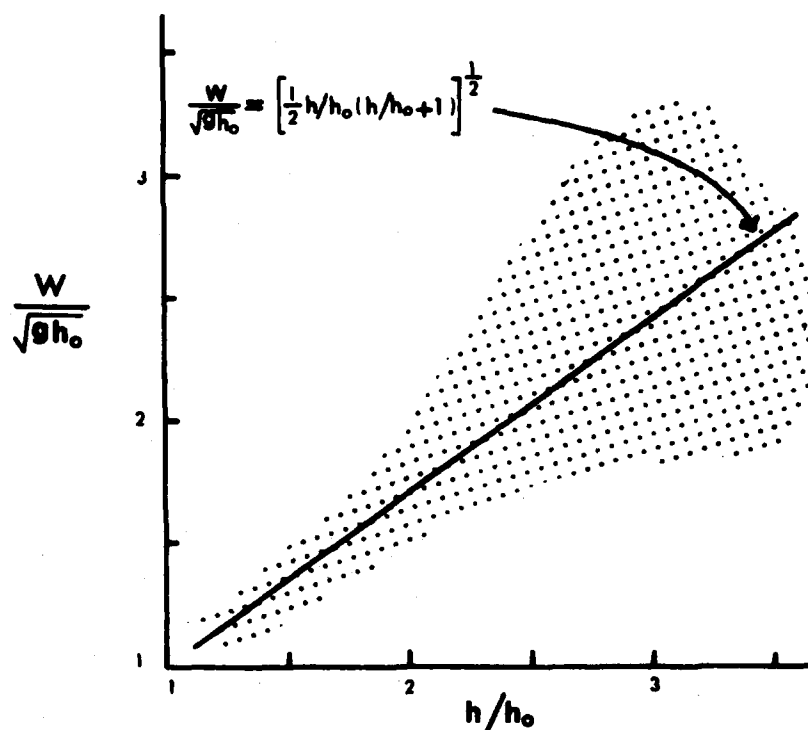


FIGURE 3.1 Diagram showing the extent of scatter observed by Miller (1968) in a comparison of measured to theoretical bore velocities in a wave tank.

Miller expresses bore velocity as:

$$W = (gh_0)^{1/2} \left[\frac{1}{2} h/h_0 (h/h_0 + 1) \right]^{1/2}$$

which is identical to equation 2.3.

The curve plots theoretical bore velocity in dimensionless form, ie.

$$W/(gh_0)^{1/2} \text{ vs } h/h_0$$

The shaded area indicates the scatter of observed bore velocities.

Note the increase in scatter as h/h_0 increases.

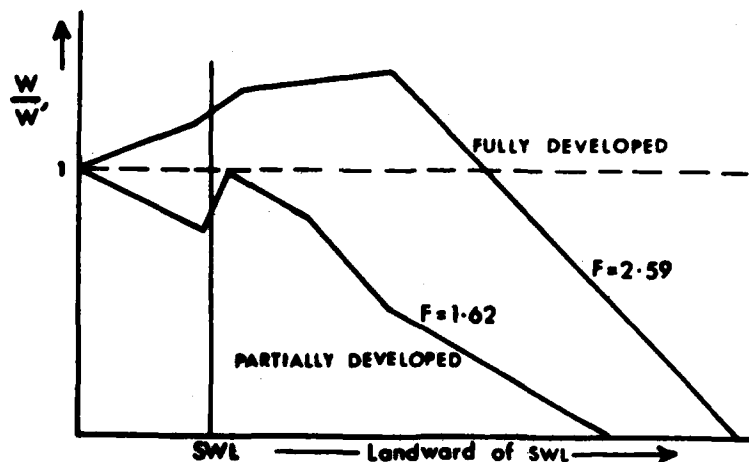


FIGURE 3.2 Example of Miller's (1968) findings for change in bore velocity across a model profile.

The diagram sketches the change in velocity of a fully and a partially developed bore as the bore climbs the slope and runs onto the beach.

The velocity of the fully developed bore increases up to and beyond the shoreline (SWL).

The partially developed bore experiences a decrease in velocity across the slope followed by a rapid increase near the shoreline.

$$F \text{ (froude No)} = W/(gh_0)^{1/2}$$

$W/W' =$ ratio of observed bore velocity to initial velocity in flat part of channel.

Data are for a 2° slope.

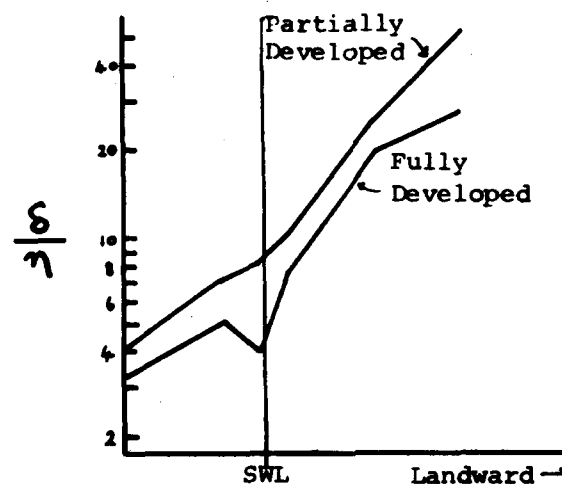


FIGURE 3.3 Example of Miller's (1968) findings for change in the shape of the bore front as the bore progresses across the slope.

The curve for the partially developed bore shows a gradual flattening of the face over the distance from inception to maximum run-up.

The fully developed bore experiences a marked steepening immediately prior to reaching the beach face (SWL), followed by a flattening as it climbs the dry slope.

δ/η is the ratio of horizontal bore face length to bore face height.

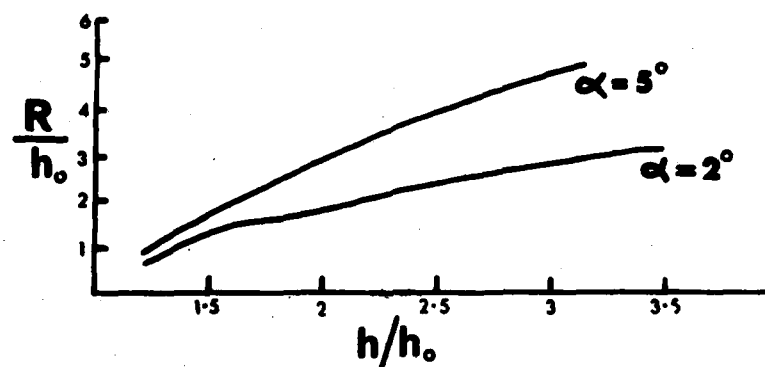


FIGURE 3.4 Example of Miller's (1968) findings for run-up height relative to bore height.

The curves lie on the observations for a 2° and a 5° slope and suggest that run-up height may be partially dependent on beach slope.

R - vertical run-up height above still water level.

h - depth of water behind bore face in flat section of channel.

h_o - still water depth in flat section of channel.

agreement between the theory and the behaviour of bores generated in the tank, especially when these were fully developed. However, the data raise some doubts concerning the usefulness of the theory as an accurate quantitative predictor. First, Miller points to the significant scatter between theoretical and observed celerities which seemed to increase with bore strength (with a cautionary note that this may be partly due to experimental inaccuracies in recording the speed of very turbulent bores). Secondly, the observations show that collapse of the bore face at the shoreline is gradual rather than instantaneous. Finally, it was found that run-up height relative to bore strength increased with increasing beach slope, contrary to equation 2.12.

Meyer (1970) attempts to trace the source of the discrepancies by applying the friction term proposed by Freeman and LeMehaute (1964) to Miller's data. This has the effect of bringing observed and estimated run-up heights more closely into line for small slopes ($<5^\circ$) but widens the gap for steeper slopes (10° & 15°). Moreover, the agreement for small slopes is reached by using a value for C_h which is 8 to 10 times larger than normally observed in hydraulics. Meyer and Taylor (1972) suggest that the effects of dissipation may be large enough to account for the discrepancies particularly in the region of bore collapse at which point there is, theoretically, a very rapid rise in bore strength and dissipation (see Chapter 2). Meyer (1970) recommends an examination of the three distinct stages in the bore-swash cycle as a way of gaining a more thorough appreciation of the factors involved:

- (i) the travel of the bore across the inner surf zone during which time the inviscid theory may be reasonably

valid,

- (ii) the bore collapse stage where turbulent dissipation may become very important, and
- (iii) the swash phase where friction with the rough bed would be expected to be the dominant force on both the run-up tip and the thin sheet of water behind it.

Miller's experimental results shed some light on the behaviour of water in each of these phases. Bore velocity, for example, is measured over the entire profile and bore collapse is described in detail for partly and fully developed bores.

Suhayda and Pettigrew (1977) examine aspects of bore theory from field data gathered using photographic techniques. Movies were made of waves crossing a surf zone (slope 0.025) from the break point to the beach face and a series of equally spaced, graduated poles were used as reference points and enabled the authors to comment on aspects of the wave motion, particularly celerity and wave height. Their data however does not bear directly on bore theory since most waves filmed appear to have been spilling breakers. The important conclusion to be drawn from the study is that bore theory is inappropriate for modelling wave motion on those parts of the profile where bores do not dominate (such as the outer surf zone).

3.2 EXPERIMENTAL AIMS

My aim in this set of experiments was to collect data from a natural beach which could be used directly to examine specific aspects of bore theory, namely:

- (i) velocity of fully or partially developed bores over

a flat surf zone,

(ii) the nature of the bore collapse at the shoreline,
and

(iii) the properties of resulting swash.

Accordingly, experiments were conducted in the extreme landward region of the surf zone where well developed bores dominate. Observations were restricted to waves with clearly distinguishable bore-like properties ie. a steep, turbulent face and very long wave length relative to water depth. On the particular beach chosen, this region was approximately 30 metres wide over which water depths ranged from zero at the shoreline to 0.5 metres at the seaward limit and beyond which waves appeared to be more closely related to spilling breakers than to bores. Under different wave and tide conditions or on a different profile the dimensions of the region would change.

3.3 FIELD SITE AND DATA COLLECTION

The experiments were conducted in early 1982 on the south coast of New South Wales at the northern end of Seven Mile Beach (Shoalhaven Bight). This beach is characterised by a flat, highly dissipative profile composed of fine quartz sand. Multiple bars often extend the full length of the beach and are best developed in the north. The wave climate of the region is dominated by moderate to high energy swell which persists throughout the year, mainly arriving from an easterly or south-easterly direction and superimposed on this is a highly variable wind wave climate (Thom et al., 1973). The tides are semi-diurnal with an average spring range of 1.6 metres. The northern end of Seven Mile Beach has a

southeasterly aspect and is subject to the full force of the dominant swell. The location and configuration of the experiment site are shown in figure 3.5.

Topographic conditions on the three days during which observations were made remained essentially the same and are detailed in figure 3.6. On the first two (consecutive) days a wide shoal was attached to the beach and partially drained on the southern end by a longitudinal channel. The slope ($\tan \alpha$) measured from the top of the swash zone to the vicinity of the break point was 0.04 while that of the inner part of the surf zone was 0.03. Breakers ranging in height from 1 to 1.5 metres plunged on the seaward edge of the shoal with a period of 10-12 seconds. All waves in the last 30 metres of the 60 metre wide surf zone had the appearance of well developed bores. Three weeks later, the channel had infilled slightly but the slope of the shoal had not changed significantly. Waves on this third day of experiments were lower (≈ 1 metre) with a period of 8 to 10 seconds.

A method which utilizes a cine-camera and a number of reference stakes driven into the sand was employed to collect all data. Similar methodology has been described by Suhayda and Pettigrew (1977).

To examine the relationship between bore velocity, bore height and water depth, four poles were arranged in a line normal to the beach. The poles were graduated in divisions of 100mm and were placed 1 metre apart. Individual bores were photographed as they travelled across the three metre wide transect at a camera speed of 18 frames per second, giving a time between successive frames of 0.055 seconds. The experimental set-up is shown in figure 3.7.

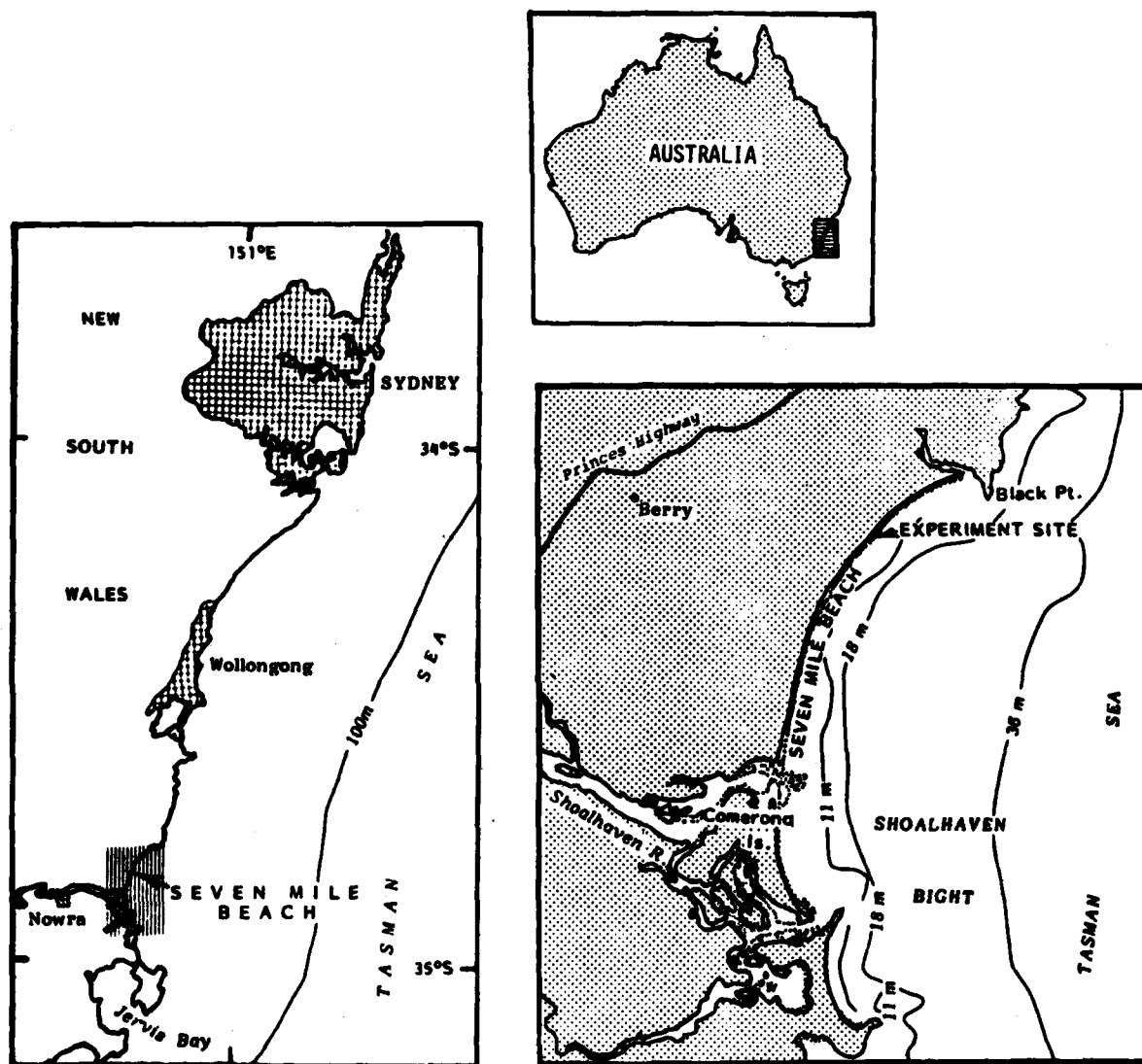


FIGURE 3.5 Location and configuration of Seven Mile Beach.

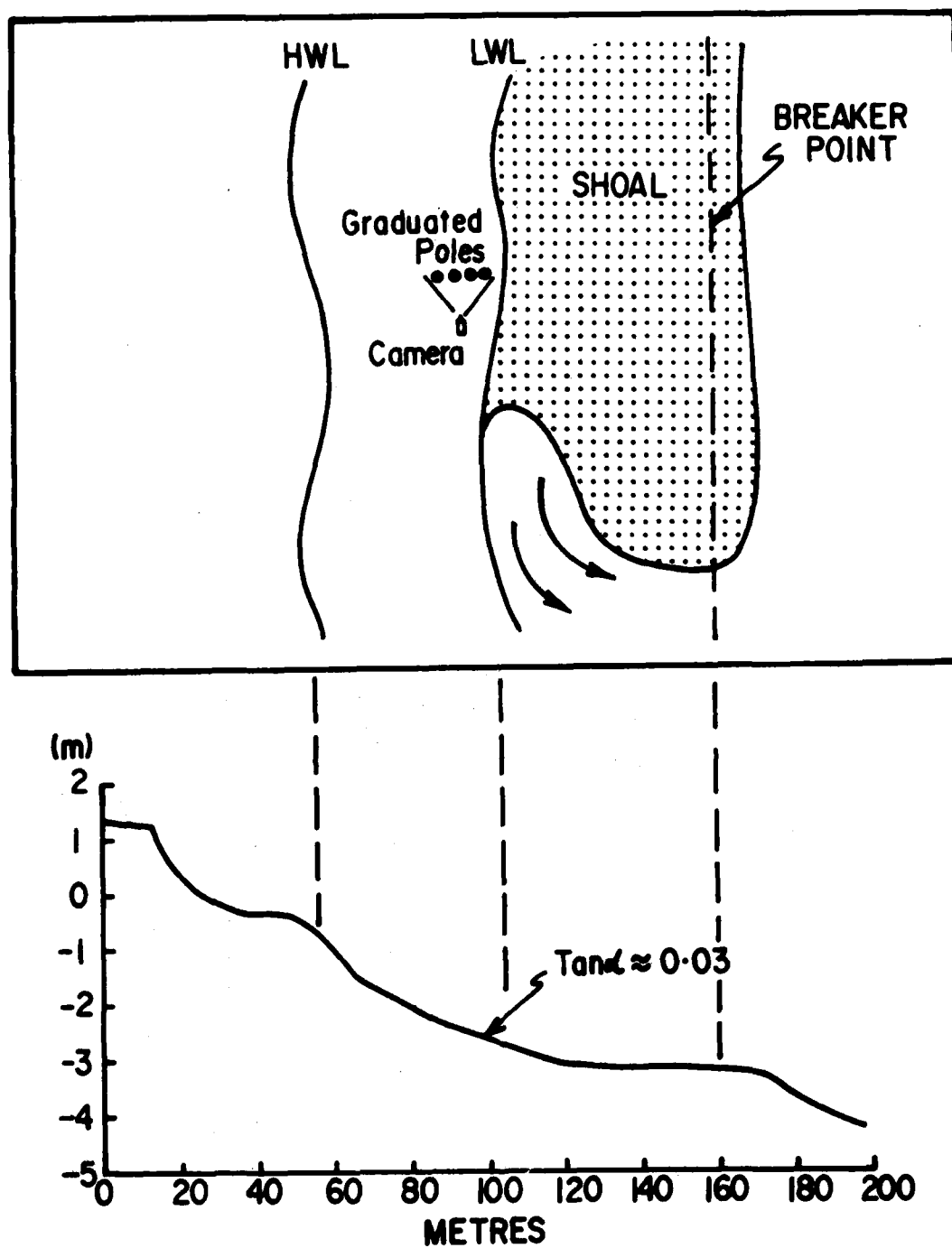


FIGURE 3.6 Configuration of Seven Mile Beach experiment site.

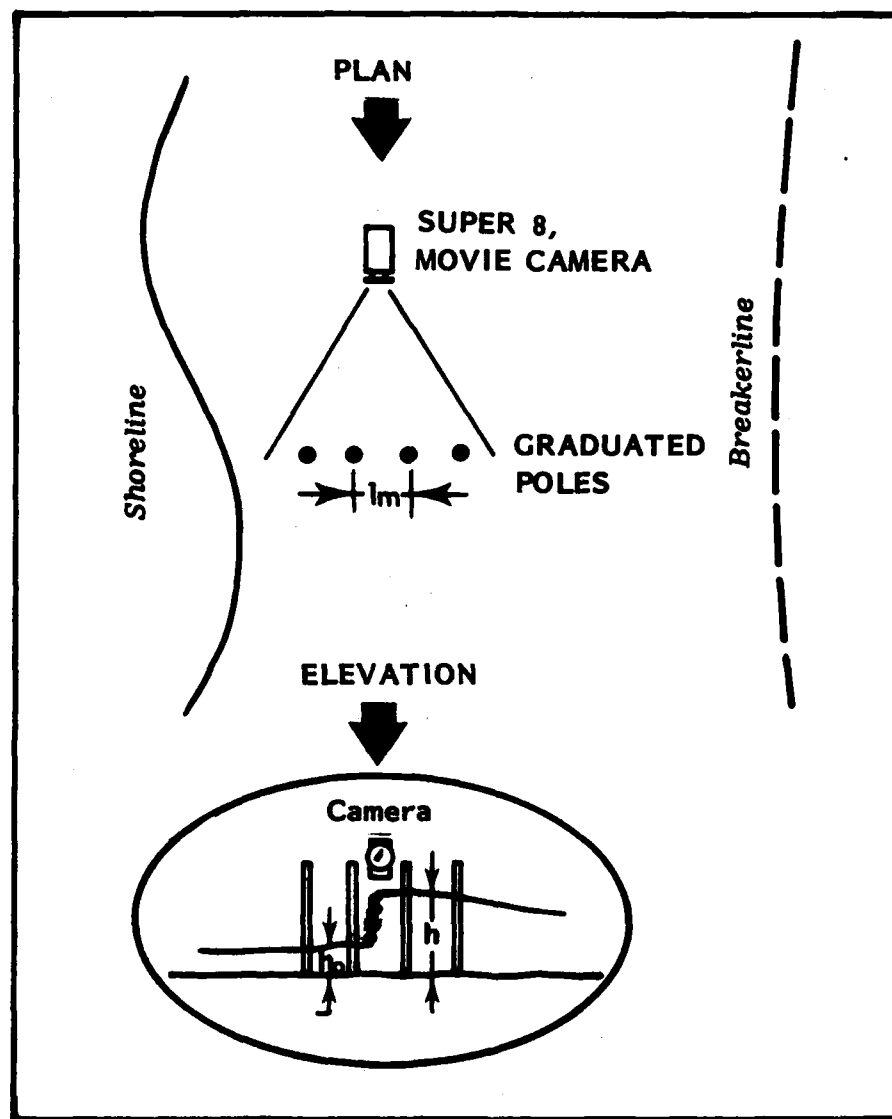


FIGURE 3.7 Experimental design for photographing bores on Seven Mile Beach.

The position of the transect was changed several times so that bore motion in different depths could be filmed. At the outer most position, water depth varied between 300 and 400 mm and bores ranged in height from 100 to 250 mm. Here the sand surface was never exposed. The inner most position was set up in the transition region between bores and swash with the dual aim of recording bore propagation in very shallow water and also capturing bore collapse and the initial stages of run-up.

A total of about fifty bores were filmed of which twenty seven have been used for analysis. In all cases, a field assistant waited by the landward most pole holding a small white float which was released as a bore approached. This release was timed roughly so that the float would traverse the poles in the return flow before the arrival of the bore. Filming commenced just prior to the release and was terminated after the passage of the bore. The movement of the float was later analysed to give an estimate of the velocity of the seaward current opposing the bore. Bores were considered unsuitable for analysis when spray and foam obscured graduations on the poles, when the float was not clearly identifiable in the film or when the bore suffered interference from excessively strong backwash or from another over-running wave.

Bore collapse and subsequent run-up proved difficult to record on film mainly because of the wide area of beach face over which the process occurred and the fact that many bores in their dying stages were significantly modified by backwash or lateral movement of water across the beach. To capture the event necessitated increasing the number of poles in the transect which in turn meant that, if graduations were still to be visible on the film,

the camera could no longer remain in a fixed tripod position. Rather, it had to be hand held by an operator who ran to keep pace with the bore as it progressed along the line of poles. More often than not the result was blurred film and only one wave was reliably recorded. An alternative, but much more time-consuming solution to the problem, would be to set up a 4-pole transect somewhere in the transition zone and film bores that looked likely to collapse across this transect, in the hope that some actually would without interference from backwash. This was not attempted.

Films were analysed by passing them frame-by-frame through a standard micro-fiche viewer. Water depths were read from the pole graduations and velocities were estimated by counting the number of frames required for a bore to travel between two poles. In the case of the four pole transects, 3 sets of consecutive measurements were usually obtained and then averaged.

The photographic method used proved to be a simple, accurate and cost effective way of collecting the type of wave data required. Manpower requirements were minimal (2-3 people) and the technique has considerable logistic and cost advantages over those based on conventional electronic recording instruments. Moreover, in addition to providing quantitative data on wave characteristics it also supplies a visual record of the progress of the wave across the slope. There is a disadvantage however, that, without the use of surface floats, no data can be extracted on water velocities on either side of the wave front. Subsurface velocities are also unattainable using this method.

3.4 RESULTS AND DISCUSSION

3.4.1 BORE VELOCITY

In order to test the applicability of equation 2.3, which gives bore velocity in terms of water depths on either side of the bore front, to well developed bores in natural surf zones a large number of bores were filmed in the manner described in the previous section. Of these, 27 were selected for analysis. Data extracted from the film are presented in Table 3.1 and include :

- . depth of water in front of the bore (h)
- . depth of water behind the bore (h_o)
- . bore front velocity (W_{obs})
- . return flow velocity (u_d)

Also contained in table 3.1 are parameters computed from these data.

Peregrine (1966) defines bores as partially developed if the height-to-depth ratio (γ) lies between 0.28 and 0.75 and fully developed if $\gamma > 0.75$. Using these criteria, most of the bores examined are fully developed with the few partially developed ones occurring in the deepest water.

Figure 3.8 shows a comparison between observed bore velocity (W_{obs}) and theoretical velocity (W_{est}) calculated using equation 2.3. Note that this equation takes account of the velocity of the water on the low side of the bore which, in the case of the data presented here, is either zero or negative (flowing seaward). The graph also gives an indication of the depth of the water (h_o) on the low side of each bore.

TABLE 3.1 Bore Observations, Seven Mile Beach.

RUN NUMBER	h_0 (m)	h (m)	η	γ	u_d (m/sec)	W_{est} (m/sec)	W_{obs} (m/sec)	W_{est}/W_{obs}
1-1	0.27	0.43	0.16	0.59	0.00	2.34	1.75	1.34
1-2	0.32	0.52	0.20	0.63	-0.27	2.32	2.02	1.15
1-3	0.34	0.59	0.25	0.74	-0.20	2.61	2.08	1.26
1-4	0.35	0.56	0.21	0.60	-0.46	2.21	1.93	1.15
1-5	0.36	0.58	0.22	0.61	-0.28	2.44	2.17	1.13
1-6	0.41	0.59	0.18	0.44	-0.36	2.30	1.87	1.23
1-7	0.37	0.58	0.21	0.57	0.00	2.70	2.57	1.05
1-8	0.38	0.66	0.28	0.74	-0.60	2.38	2.13	1.12
1-9	0.42	0.64	0.22	0.52	-0.50	2.31	2.17	1.07
1-10	0.16	0.38	0.22	1.38	-1.13	1.38	1.52	0.91
1-11	0.07	0.30	0.23	3.29	0.00	2.79	2.60	1.07
1-12	0.15	0.35	0.20	1.33	-1.10	1.29	1.70	0.76
1-13	0.11	0.35	0.24	2.18	-0.90	1.78	1.50	1.19
1-14	0.07	0.21	0.14	2.00	-1.00	1.03	0.87	1.18
1-15	0.04	0.20	0.16	4.00	0.00	2.42	2.08	1.17
1-16	0.21	0.33	0.12	0.57	-0.82	1.22	1.17	1.04
1-17	0.11	0.36	0.25	2.27	-1.20	1.55	1.45	1.07
2-1	0.10	0.28	0.18	1.80	-0.56	1.72	1.50	1.15
2-2	0.15	0.36	0.21	1.40	-0.82	1.63	1.50	1.09
2-3	0.20	0.40	0.20	1.00	-1.13	1.29	1.29	1.00
2-4	0.09	0.24	0.15	1.67	0.00	2.08	1.88	1.10
2-5B	0.30	0.55	0.25	0.83	-0.95	1.81	1.80	1.01
2-5C	0.21	0.50	0.29	1.38	-0.95	1.93	1.80	1.07
2-6B	0.25	0.45	0.20	0.80	-0.30	2.18	2.00	1.09
2-6C	0.20	0.40	0.20	1.00	-0.30	2.12	1.80	1.18
2-11	0.10	0.40	0.30	3.00	0.00	3.13	2.86	1.09
2-12	0.18	0.32	0.14	0.78	-0.40	1.69	1.53	1.10

h_0 - Depth of water in front of bore

h - Depth of water behind bore

η - height of bore face ($h - h_0$)

γ - height-to-depth ratio (η/h_0)

u_d - velocity of water in front of bore
(minus sign indicates seaward flow)

W_{est} - bore velocity estimated using eq. 2.3

W_{obs} - observed bore velocity

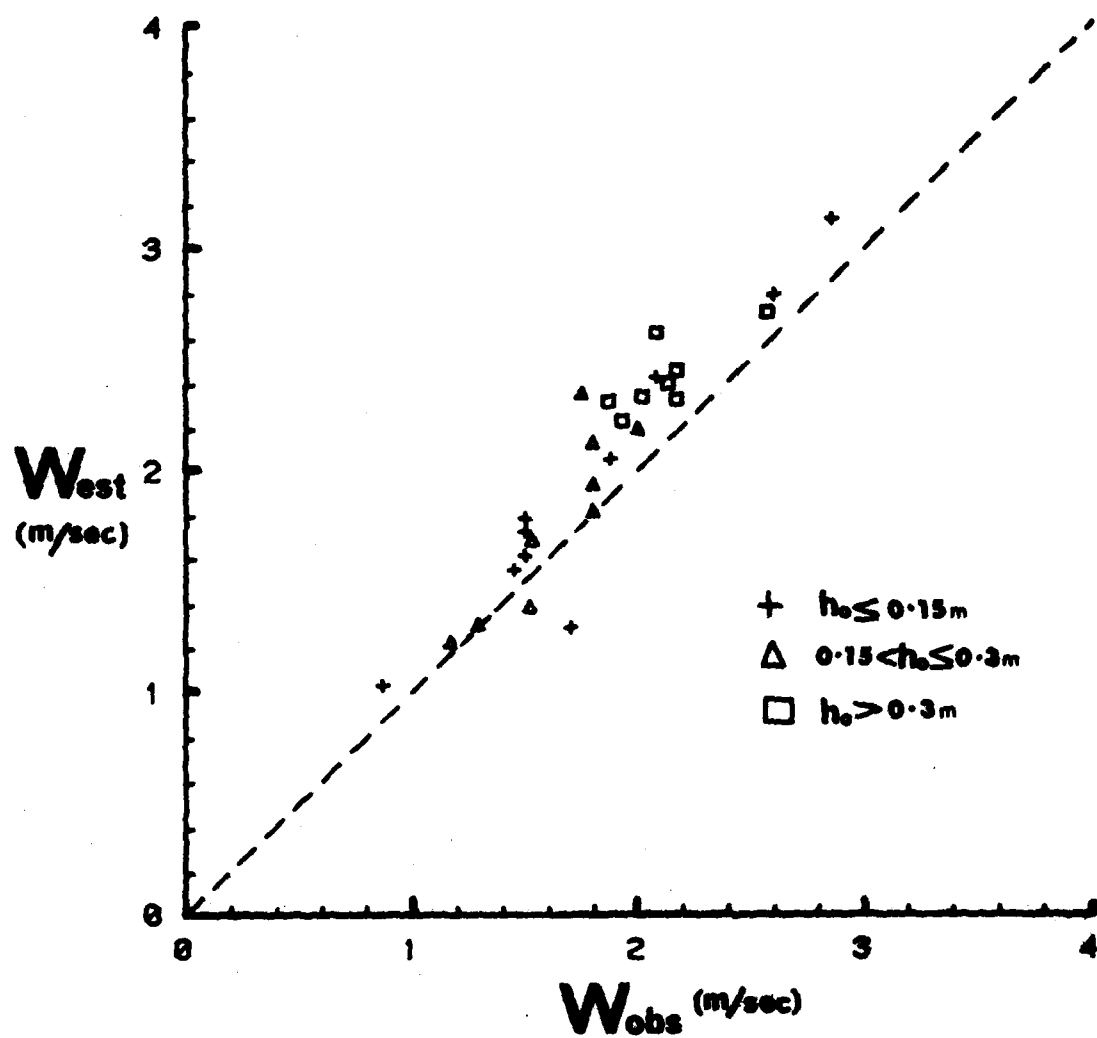


FIGURE 3.8 Comparison of observed bore velocities (W_{obs}) with velocities estimated using equation 2.3 (W_{est}).

Data are from Seven Mile Beach, NSW.

Number of observations - 27.

Symbols show depth of water in front of each bore (h_o).

The proximity of most points to the line of perfect correlation in figure 3.8 suggests that equation 2.3 adequately models velocities of fully and partially developed bores over a range of depths and also, that a simple subtraction of velocities is sufficient to cope with the interaction of a bore propagating over a seaward moving body of water. Figure 3.9 indicates that there is no relationship between return flow velocity and proximity of data points to the diagonal in figure 3.8 and on this basis it could be argued that the interaction process is linear. This may be so for the range of velocities observed here but return flow velocities substantially higher are common on flat beaches and often result in the arresting of bore motion. The interaction process may become more non-linear due to increased turbulence as return flow velocities necessary for bore suspension are approached.

A noteworthy feature of figure 3.8 is that a degree of theoretical overprediction remains even after allowing for return flows. This may reflect the fact that the theory does not account for energy losses due to bottom friction and to turbulence in the bore face. However, the data presented here suggest that the omission does not detract significantly from the model's usefulness for predicting bore velocities in the inner surf zone. Miller (1968) calls for a re-formulation of the theory to include friction and turbulence but this has not yet been done satisfactorily. At this time, the roles of friction and turbulent dissipation in the surf zone are poorly understood.

It must be pointed out that return flow velocities used in equation 2.3 are probably underestimated due to

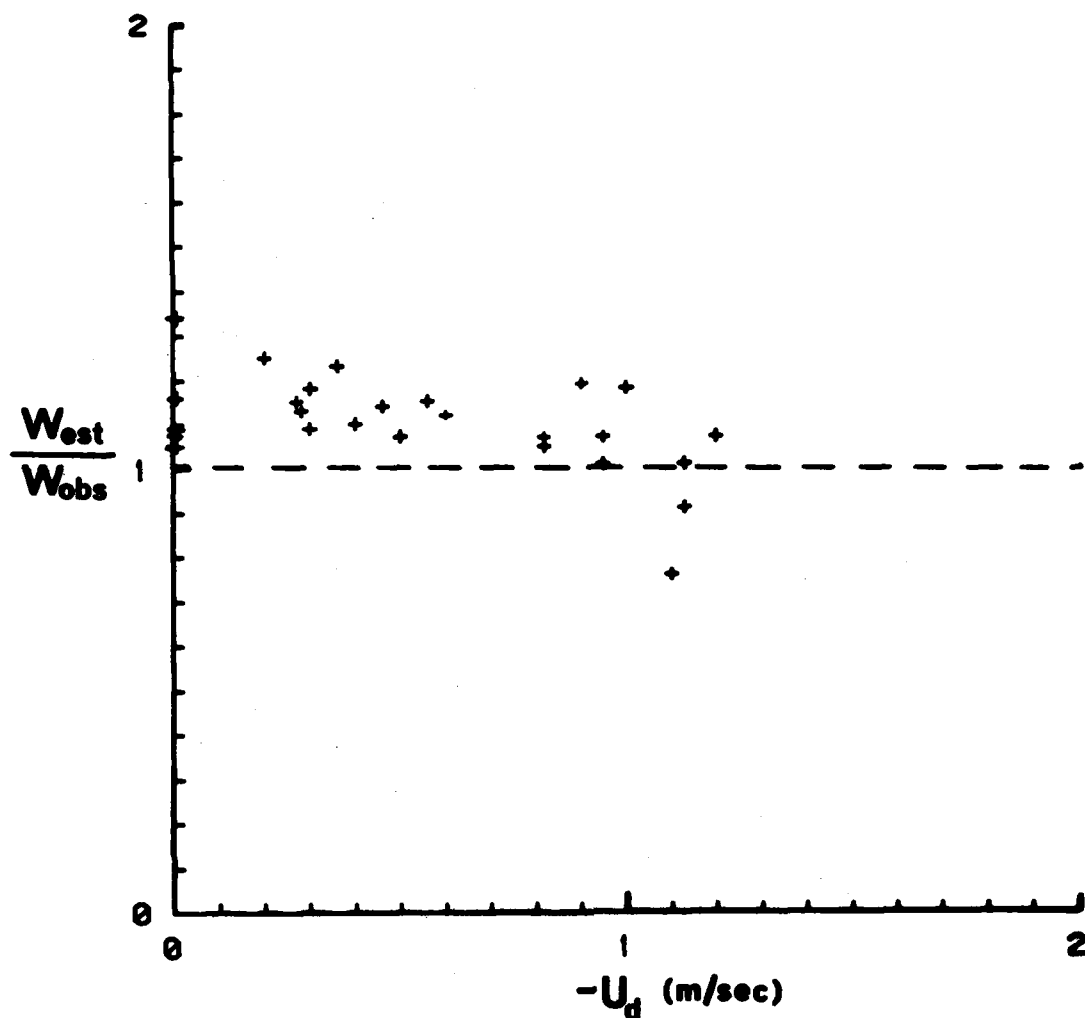


FIGURE 3.9 Velocity of water on low side of bores ($-U_d$) measured on Seven Mile Beach graphed against the ratio of estimated to observed bore velocities (W_{est}/W_{obs}).

The minus sign in ($-U_d$) indicates seaward flowing water.

the fact that the measurements were, by necessity, made before the arrival of the bore at the line of graduated poles. The analysis of the films revealed that the measuring float often accelerated across the transect, and this acceleration no doubt continued, in some cases, until bore arrival. More accurate measurements of return flow may further reduce the theoretical overprediction evident in figure 3.8.

3.4.2 SHOREWARD CHANGES IN BORE CHARACTERISTICS

Keller et al. (1960), state that bore velocity will increase towards the shoreline as a result of increasing bore strength, and this is verified by Miller (1968) for the case of fully developed bores (figure 3.2). Miller's data show that partially developed bores experience a drop in velocity across the surf zone followed by a sharp rise just before the shoreline and this is also noted in the field by Suhayda and Pettigrew (although the latter point out that they were dealing with 'unborelike' waves for the most part).

The present study yields little direct data on the change in bore velocity across the inner surf zone because of the fact that the bores rarely propagated into stationary water, and the measurements necessary to provide data on flow direction and velocity of the underlying water body were not obtainable using photographic techniques. It is significant however, that the ratio of bore height to water depth increased with decreasing water depth (figure 3.10). If h_0 is used as an indicator of proximity to the shoreline, this implies a shoreward increase in the bore height-to-depth ratio as predicted by the theory.

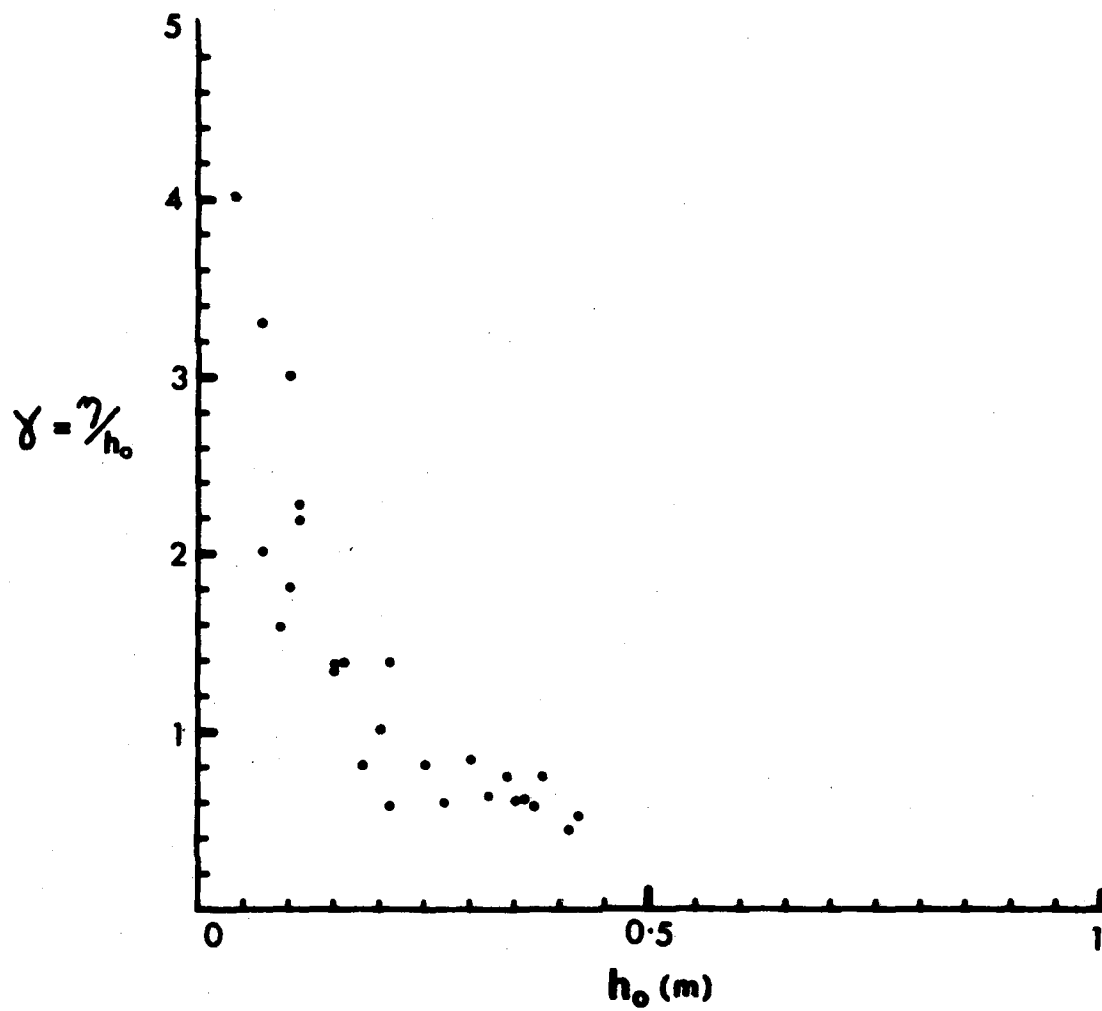


FIGURE 3.10 Bore height-to-depth ratio (γ) for bores measured in different depths of water (h_0).

3.4.3 BORE COLLAPSE AND RUN-UP

The nature of bore collapse and run-up at the shoreline on a flat beach are influenced and modified to a large degree by backwash effects and by overtaking bores. The zone over which the transition between well developed bore and swash occurs is often wide and the behaviour of individual bores hard to predict. For this reason, and those outlined in section 3.3, the film data are not comprehensive in this area. One section of film however, does clearly show a bore collapsing within the range of the graduated poles and careful observation of the inner surf zone suggests that the sequence now described is typical of the events that occur when a small bore (height $< 0.2\text{m}$) arrives at a stationary shoreline ie. after backwash is complete.

Figure 3.11 A small bore approaches the transect upon

- (a) - (b) which is a very thin film of water
(the transect lies below the intersection of the water table with the sand surface and is permanently saturated).

The shoreline is approximately level with pole 1. One metre seaward (water depth $< 10\text{mm}$) the steep bore face begins to flatten and has almost disappeared by the time the leading edge reaches the first pole. Approximate average velocity of the leading edge over the zone of collapse = 2.25 m/sec .

- (b) - (c) Bore face completely disappears. Leading edge accelerates to a velocity of $\approx 3\text{ m/sec}$.

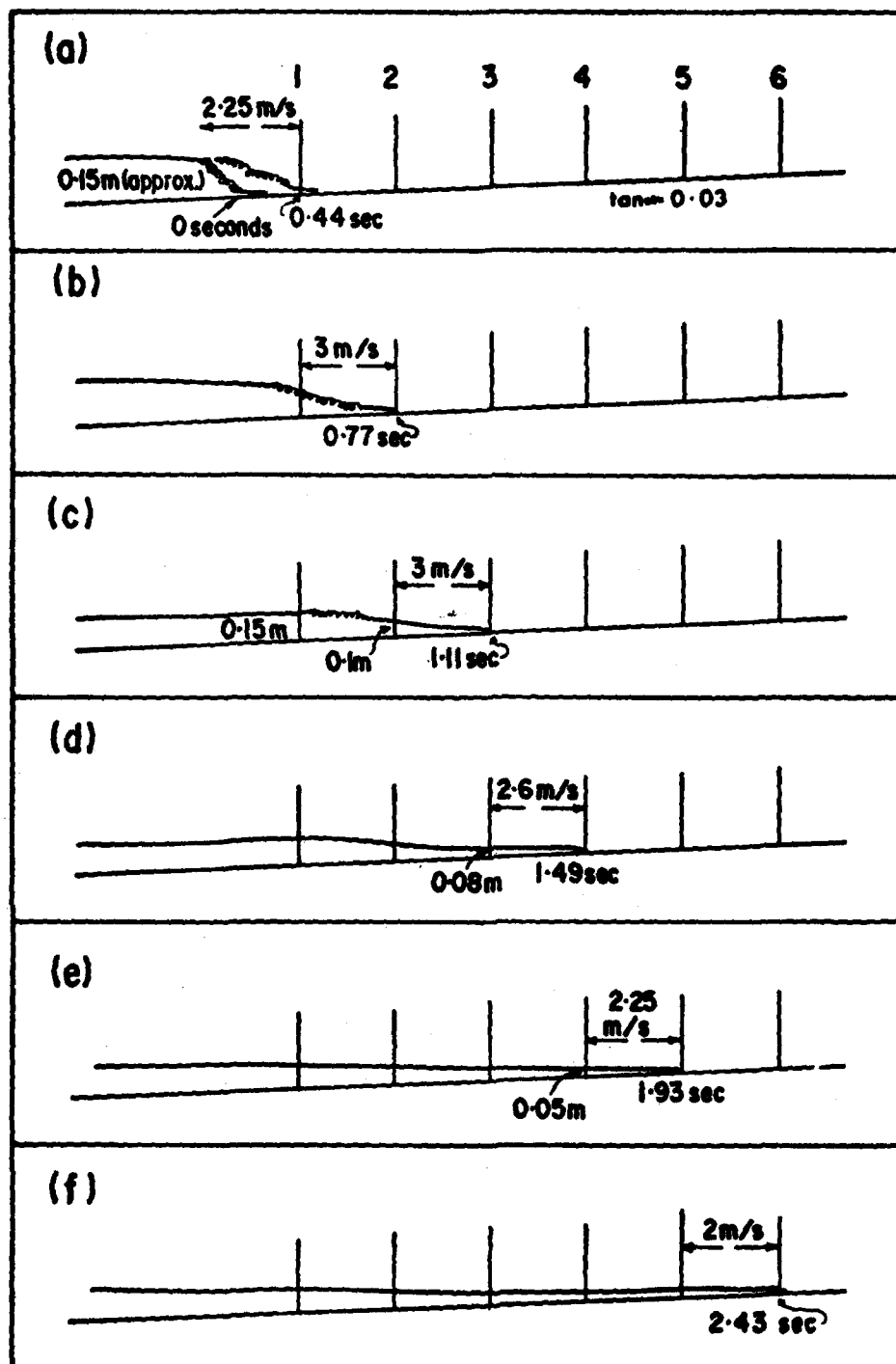


FIGURE 3.11 Example of bore collapse and run-up sequence on a flat beach.

Diagram shows the velocity at the base of the bore during collapse and leading edge velocities of subsequent run-up.

Approximate depth of run-up lens in the initial stages is also shown.

NOTE: DIAGRAM IS NOT TO SCALE IN THE VERTICAL.

- (c) - (f) Leading edge moves up slope with decreasing velocity. The swash front is bore-like in appearance and is highly aerated. Foam covers the surface of the entire swash lens.

The observations show both a rapid disappearance of the steep bore face and an acceleration of the leading edge of water at the shoreline. This is consistent with the theoretical predictions outlined in chapter 2 and also with Miller's (1968) findings for fully developed bores.

It is interesting to compare the velocity of the leading edge of water after bore collapse with that predicted for a dam break. Stoker (1957) shows that water will flow from a broken dam onto a dry bed with a leading edge velocity of $2\sqrt{gh}$, where h is the depth of water in the dam. In this case, the velocity immediately after collapse is approximately 3.0 m/sec which is greater than $2\sqrt{gh}$ for $h=0.15\text{m}$ (from figure 3.11). On the other hand, the observed velocity is less than that given by $U_0 = u' + 2\sqrt{gh}$ (eq. 2.6) for $u' = 2\text{ m/sec}$, the latter being an approximation of the water velocity near the bore front a short time before collapse. Note that the data do not yield values for W and h at some small h_0 (just prior to bore collapse) which would enable a better estimate to be made of u' using equation 2.4.

The simple equations describing the motion of the leading edge after bore collapse can be tested by applying the data shown graphically in figure 3.11 to equation 2.7. Assuming a value for U_0 of 3 m/sec, a beach slope of 0.03 (see figure 3.6) and a travel time between poles 1 and 6 of 1.99 seconds, the equation gives an expected velocity at pole 6 of 2.4 m/sec. The actual average velocity over

the 1 metre distance between poles 5 and 6 was 2 m/sec, indicating that friction is probably considerably important even in a situation where run-up travels over a saturated ('slick') sand surface.

The following set of general observations were made during the course of filming. They indicate the range of situations common in the bore-swash transition zone on a flat profile and may serve as a guide for future, more quantitative observations.

Small bore (<0.2m) - rapid collapse at the shoreline in
Stationary shoreline the manner described above.

Very small bore - transition much more difficult to
(<0.05m) observe. Appears to be gradual.

Seaward moving - progress of bore front is
shoreline arrested and bore height
(backwash) gradually decreases. Rapid
collapse occurs at completion
of backwash.

Sustained backwash - bore height is reduced to zero.
No run-up is generated.

Sustained strong - stationary hydraulic jump is
backwash generated in the region of the
bore front. Much sediment
entrainment.

Shoreward moving
shoreline

- bore over-running a moving swash lens experiences a rapid collapse as soon as the leading edge of the swash is reached. Accelerations appear large and maximum run-up penetration is achieved.

3.5 SUMMARY

Detailed analysis of several photographic records of waves with gross bore characteristics shows that their velocity in shallow water ($<0.5\text{m}$) is correctly given by the theoretically derived bore equation (2.3).

Bore height does not appear to be limited by depth. Rather, the bore height-to-depth ratio (γ) appears to increase with decreasing water depth over the inner-most part of the surf zone. This is consistent with the theoretical predictions of Keller et al. (1960).

More general qualitative observations suggest that bore collapse (for all but the smallest bores) is rapid, occurring as soon as water depth reaches (or becomes close to) zero, and is followed by an acceleration of the leading edge of water. (The critical minimum depth for the initiation of collapse is not known). The behaviour of the bore near the shoreline under most natural conditions is not easily identifiable because of the strong modifying influence of backwash.

CHAPTER 4

OBSERVATIONS OF BORES AND RUN-UP ON STEEP BEACHES.

Bores on steep beaches are not as strikingly evident as are their fully developed counterparts on flat beaches. Nevertheless, observations suggest that parts of the model outlined in Chapter 2 may be relevant to the study of steep beach processes. Note the similarity between the definition diagram in figure 2.3 and the example from nature of a bore on a steep beach (figure 4.1)

In this chapter it is argued on the basis of visual observations that bores form on steep profiles. Bore types are discussed in relation to breaker types and detailed observations are then presented for major bores which deal with the bore collapse stage, swash velocities and swash excursion lengths.

4.1 DATA SOURCES

Cine-films have been made of wave action on many steep profiles on the New South Wales coast and analysis of these has yielded the observations and data which follow.

Several sets of films have been used. Most were made on steep beaches as part of larger experiments which



FIGURE 4.1 A major bore approaching the shoreline
on a steep beach.

involved computer controlled data logging of wave and current metering instrumentation deployed in the surf zone and on the beach face. The instrumentation system used has been described by Bradshaw (1978) and details of the experiments have been published by Wright et al. (1979) and Bradshaw (1980). The camera was synchronised with the computer so that one frame was exposed each time the instrument outputs were sampled, the usual sampling rate being one second. Films span several 15 minute recording periods which represents several hundred waves. Reference poles were usually located 5 metres apart and the filming angle was often oblique. These limitations, combined with the relatively long interval between frames make the films unsuitable for detailed analysis of bore and swash behaviour. However, they have been used extensively to examine the relationship between breaker type and bore formation.

To provide more detailed information on steep beach processes, cine-films were made at Warri Beach on the south coast of New South Wales (figure 4.2). This two kilometre long embayed beach displays complex rhythmic inshore topography in the south and central sections but is usually steep with a plane inshore profile at the northern end. Sediments are medium coarse along the length of the beach. The experiment was conducted at the extreme northern end on a slope ($\tan \alpha$) of 0.158. Wave period was 10-11 seconds and breaker height at the shore ranged from 1 to 1.5 metres. The beach was pegged with graduated poles from 4 metres beyond the step to the top of the active berm in 2 metre increments. Only selected waves were filmed and field assistants made simultaneous records of breaker height, run-up duration and run-up width. The intention was to film only waves experiencing minimal

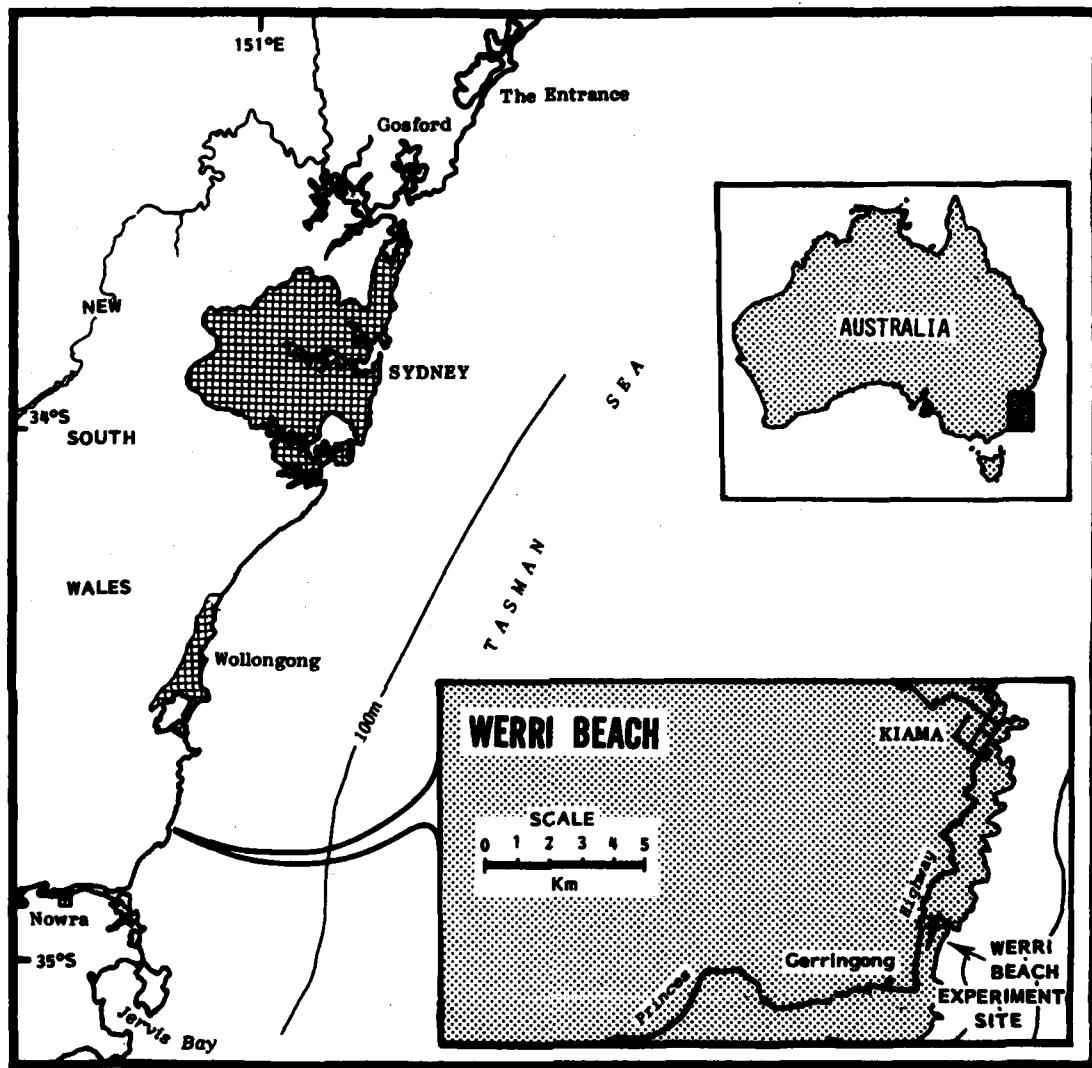


FIGURE 4.2 Location of Werri Beach experiment site.

interference from backwash; however the prediction of a 'clean' break proved difficult and subsequent viewing showed that, in many cases, the aim had not been realised. Ten were carefully selected for analysis and results are presented in section 4.3.

4.2 BREAKER AND BORE TYPES

Three types of waves are commonly observed on steep beaches; surging, plunging and collapsing. Galvin (1972) notes a relationship between breaker type and wave energy, with the progression being from surging to plunging as energy increases.

On any given beach, all three may be represented in the short term mainly because wave height and steepness are variable within the wave train. However, it would also be reasonable to expect breaker characteristics to be influenced to some degree by backwash. Through this mechanism, irregularities in breaker period (which will determine the extent and frequency of wave-backwash collisions) will add to the short term variability of breaker type.

Observations show that both major and minor bores form at the base of steep beaches. The terms are defined by Amein (1966) and are reviewed in section 2.3. Briefly, a major bore is one in which the ratio of bore height to the height of the associated wave crest is greater than 0.5. The following section examines data for this type of bore.

In general, plunging breakers are associated with

major bores at the shoreline. An illustration of this is given in figure 4.1. A simplified breaker-swash sequence is shown in figure 4.3. In the absence of powerful backwash, high waves will plunge slightly seaward of the beach-face into shallow water. A strong bore forms and traverses the short distance to the dry slope where it begins to lose its vertical face. To conserve mass as the bore moves shoreward, most excess height in the body of water behind the bore will disappear so that a major bore forms. According to Amein's (1966) analysis, the velocity of the initial run-up (and hence the distance travelled up the slope) will be influenced only by wave elements in the region of the bore face.

Surging and collapsing breakers are more readily associated with minor bores because of the high body of water that looms behind the bore face. An example is shown in figure 4.4. In the case of a minor bore, Amein (1966) shows that the run-up issuing from the bore face is quickly overtaken by wave elements in the vicinity of the wave crest.

The sequence for a collapsing breaker is shown in figure 4.5. The difference between it and the plunging breaker is that, even though a plunge may occur and form a vertical bore face, it does so well below the level of the wave crest (Galvin, 1972).

The simplified descriptions of breaking and bore development given above are complicated in nature by waves interacting on and near the beach face. The extent and frequency of the wave-backwash collision at the base of the beach, which is determined by the period of the breakers relative to swash-backwash duration, is a

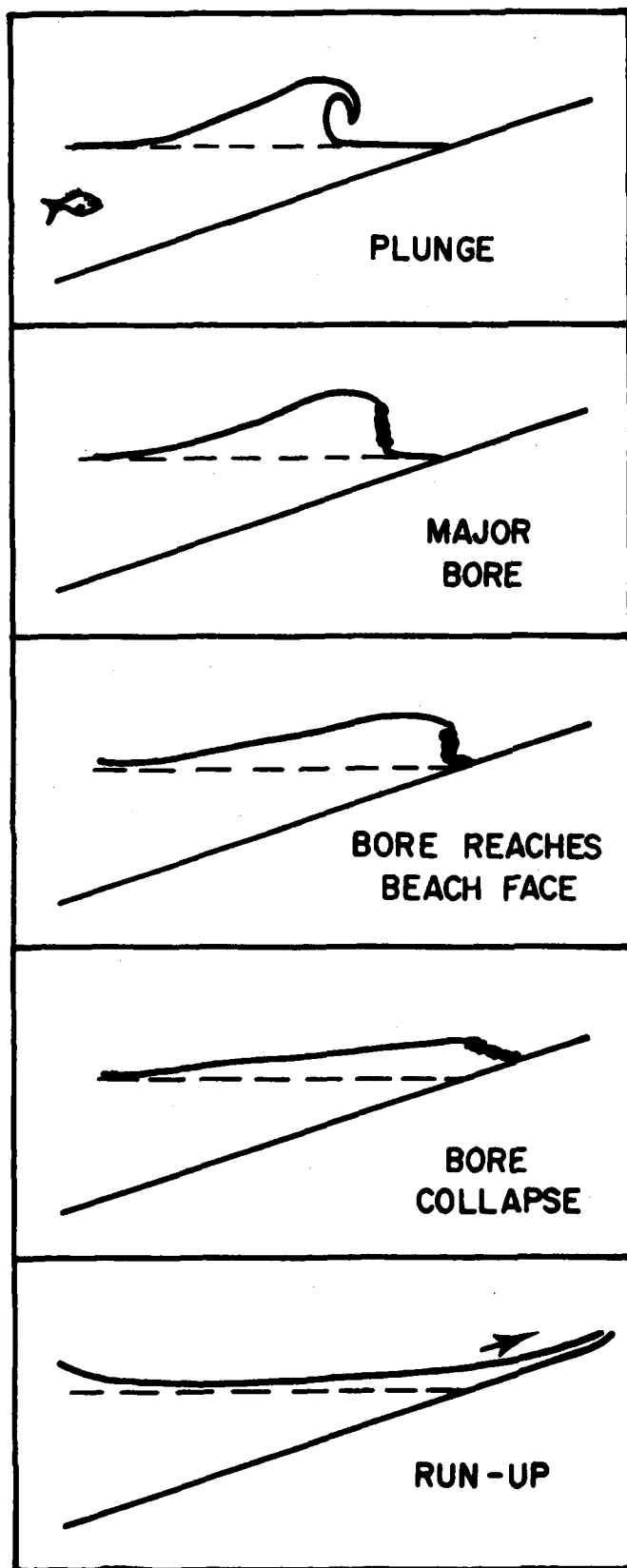


FIGURE 4.3

Formation of a major bore from a plunging breaker on a steep beach.

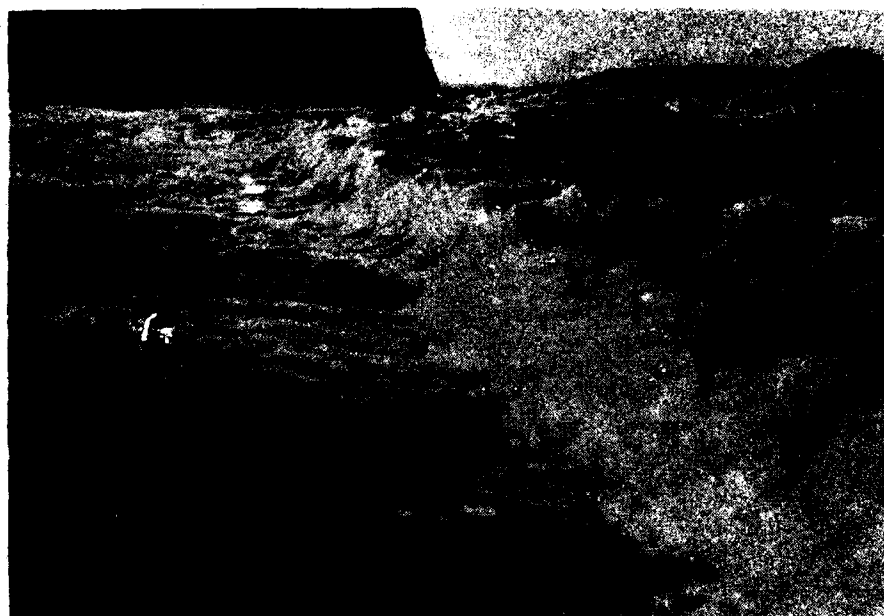


FIGURE 4.4 A minor bore at the shoreline of a steep beach. (Photo. by P. Cowell)

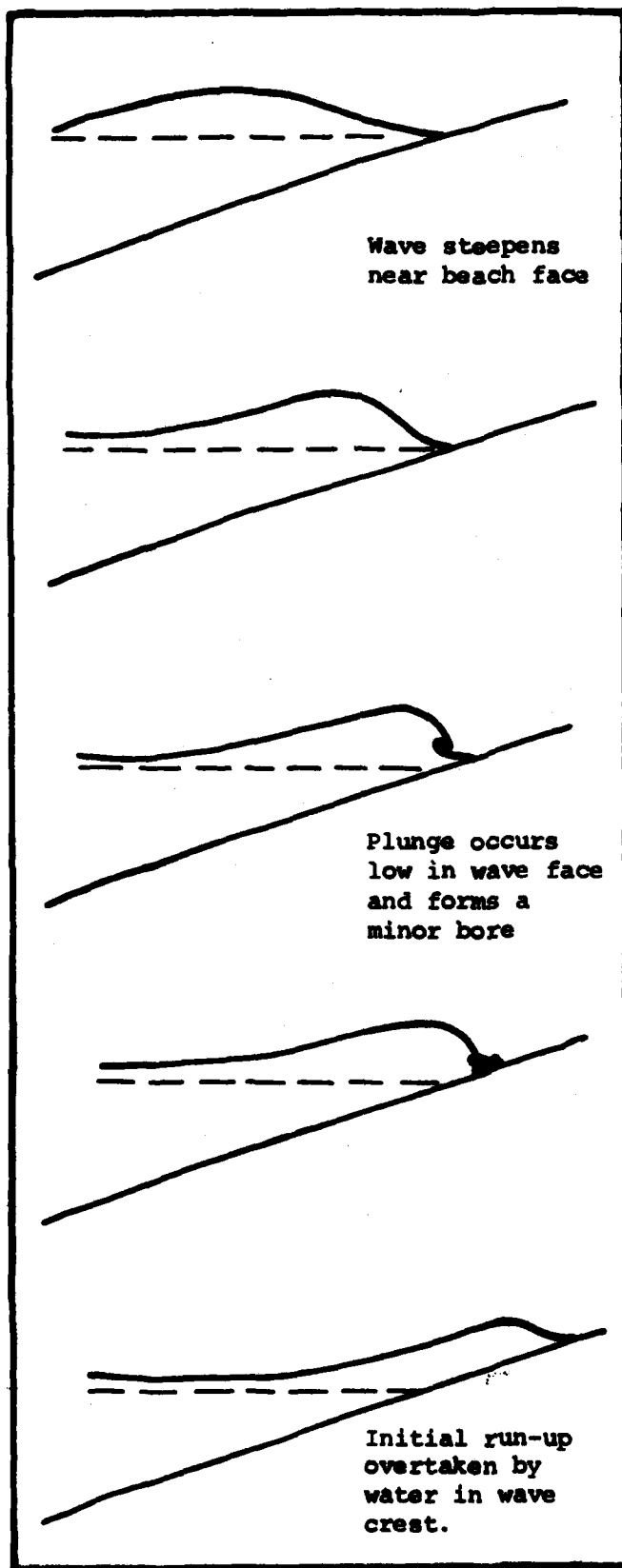
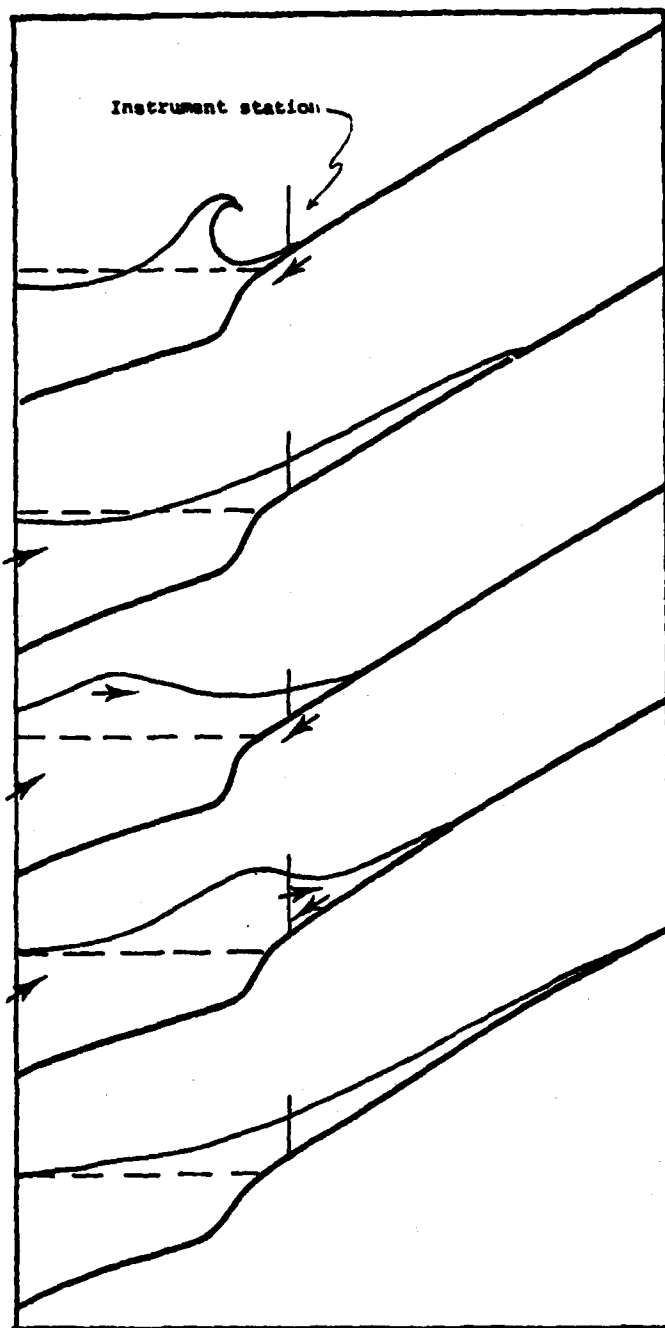


FIGURE 4.5

Formation of a minor
bore from a collapsing
breaker on a steep
beach.

significant factor. A weak backwash (or one that is almost complete before the onset of the next wave) will sometimes have minimal effect on the bore as it flattens and turns into swash. However, in the extreme case, a strong, sustained backwash will totally dissipate a bore in a violent churning action in the vicinity of the step and run-up will be insignificant.

On the other hand, the presence of a lens of water on the beach can in some cases lead to high run-up, even if the water has begun to move seaward. This is often the case when breaker period is short. The second of two closely spaced waves will often fail to plunge or collapse but instead will surge over the considerable volume of water cast onto the beach by its predecessor and will penetrate far up the slope. A succession of small closely spaced waves can often maintain a large depth of water over the step resulting in greater overall swash penetration than would be achieved by any single large wave. The process is illustrated in figure 4.6 and can also be seen in a section of chart record from a wave sensor and a flow meter located on the upper level of the step of a steep beach (figure 4.7) (this record was collected during experiments referenced in section 4.1). Troughs in current and water surface records at A and A' correspond to a strong backwash, unimpeded by incoming waves, which leaves the lower beach face exposed. Between A-A' and B-B' a wave breaks and surges up the beach and registers as a strong landward current and a rise in water level over the step. Current reversal begins at B as water drains seaward. However, its progress is checked at the bottom of the beach by the head of water of a newly arrived wave and the effect is to maintain the depth of water over the step at almost its original level (B' to



First wave plunges on dry beach after completion of previous backwash and initiates run-up.

Depth of water maintained on lower beach by backwash and by second wave.

Second wave surges over large volume of water on lower beach face resulting in high run-up

FIGURE 4.6 Sequence showing high run-up resulting from closely spaced breakers.

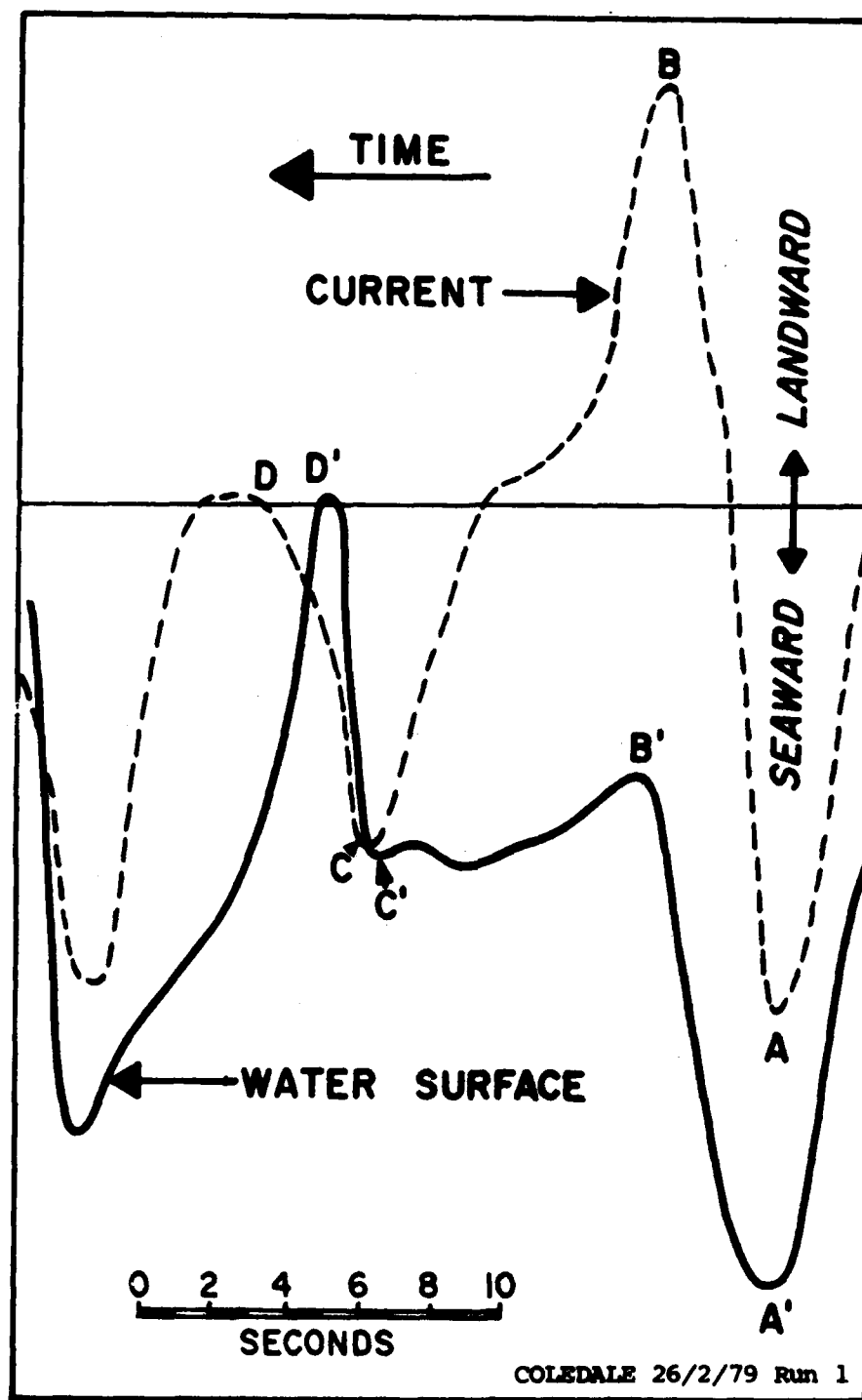


FIGURE 4.7 Current and water surface records from the lower section of a steep beach. Instruments were located slightly landward of the step (see fig. 4.6)

C'). The second wave surges onto the beach over the water remaining from the first and depth over the step doubles before backwash begins at D'.

4.3 OBSERVATIONS OF MAJOR BORES AND RUN-UP

The discussion of bore formation on a steep slope in the previous section and, in particular, the plunging breaker-bore sequence shown in figure 4.3, suggest the relevance of the bore - run-up model to this type of beach.

The model is in three parts. The first deals with the motion of a bore travelling across a gently sloping bottom, the second with the disappearance of the bore at the shore line and the third with the the run-up phase. Figures 4.1 and 4.3 indicate that the bore phase on a steep beach is of short duration - the bore is intimately bound up with the plunging breaker and travels a very short distance before encountering the dry beach. Thus, the first part of the model, which is embodied in equations 2.3 and 2.4, is inappropriate in this case. However, parts two and three may be applicable and data are now presented to examine this.

The objective of the experiment at Warri Beach was to film waves which displayed clear characteristics of major bores and which reached the shoreline with minimal interference from previous backwash. Ten waves were selected for detailed analysis and yield data on the leading edge velocity during and immediately after the arrival of the bore at the beach face, and on the width of the resulting swash lens.

The results are plotted in figure 4.8 (a & b) which show for each wave:

- (i) the approximate position of the plunging breaker at the base of the slope,
- (ii) the approximate height of the breaker. This was estimated by an assistant who stood at the shoreline and sighted the wave crest along graduations on a surveying staff,
- (iii) the zone of transition over which the steep bore face flattens and turns to run-up,
- (iii) the average velocities of the leading edge of water between reference poles (spaced at a two metre interval across the profile),
- (v) the maximum width of the run-up lens, and
- (vii) the position of the intersection of the water table with the beach face. Below this, the sand is permanently saturated.

4.3.1 BORE COLLAPSE AND LEADING EDGE VELOCITIES

In all cases shown in figure 4.8 the high vertical bore face begins to flatten as it moves into zero water depth, and completes the transition from bore to swash over a distance of between 3 to 5 metres. This transition was accompanied by an acceleration of the leading edge and by much turbulence.

Table 4.1 gives the velocity of the leading edge of water at the end of the collapse phase for each wave. It is noteworthy that all are high compared to swash flow velocities reported hitherto in the literature (see summary of reports of swash zone flow velocities in Kirk, 1975, p120) and this is partly due to the fact that

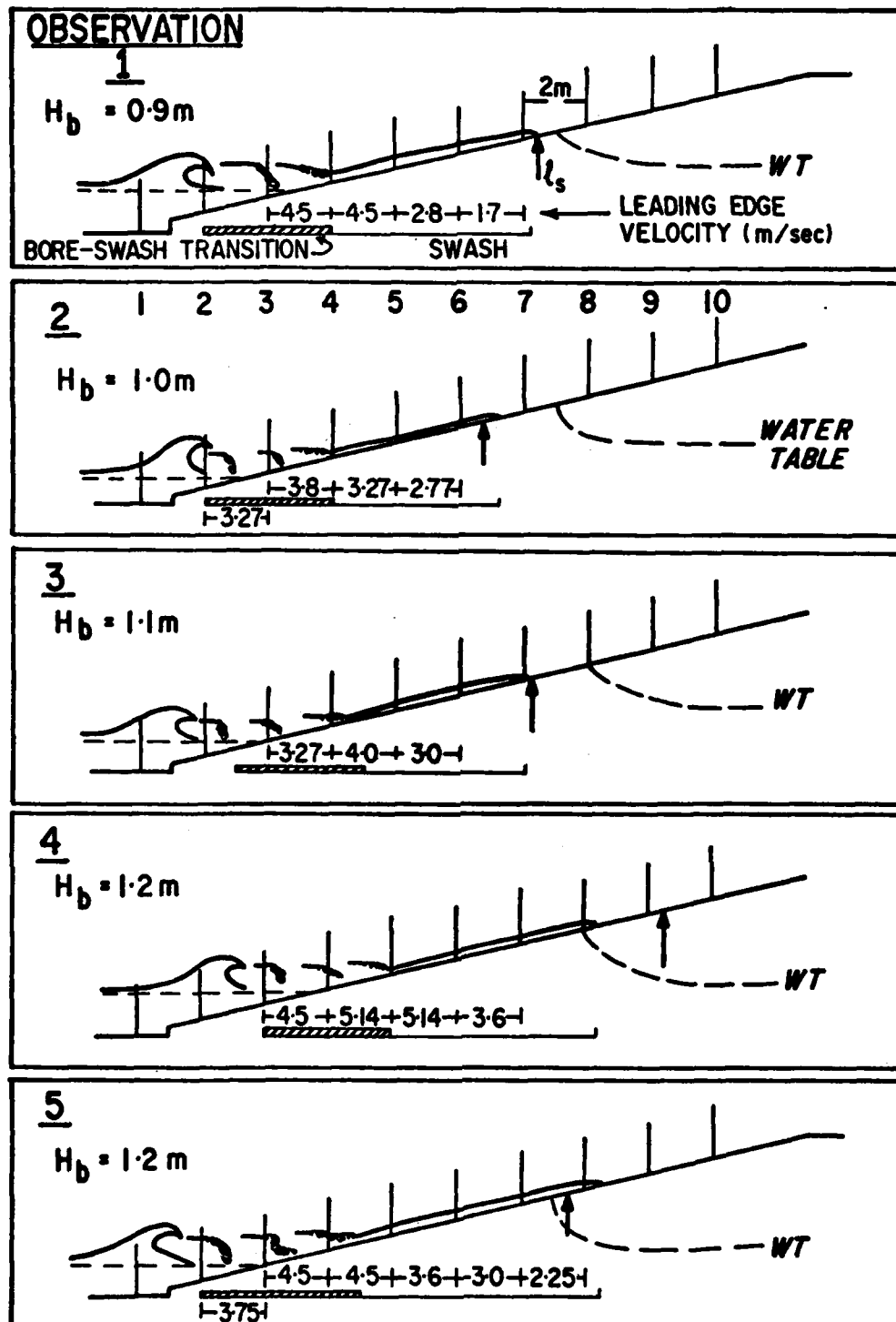


FIGURE 4.8a Werri Beach bore-swash observations waves 1 to 5.

H_b - breaker height

Arrow indicates maximum theoretical run-up (see text).

NOTE: DIAGRAM NOT TO SCALE IN THE VERTICAL.

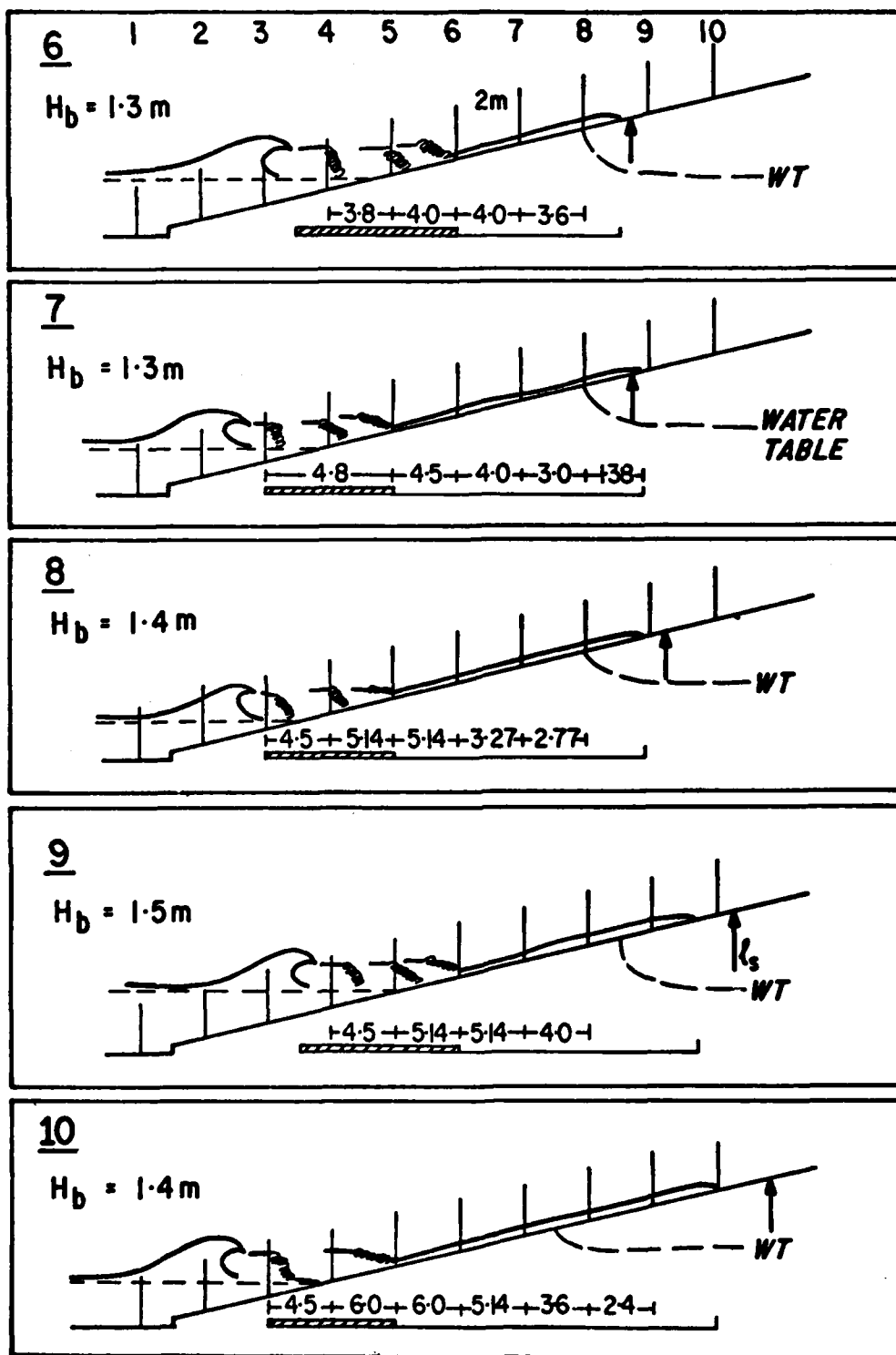


FIGURE 4.8b Werri Beach bore-swash observations waves 6 to 10.

OBSERVATION NUMBER	BREAKER HEIGHT H_b (m)	LEADING EDGE VELOCITY AT END OF BORE COLLAPSE U_o (m/sec)	$k = \frac{U_o}{\sqrt{gH_b}}$
1	0.9	4.5	1.51
2	1.0	3.8	1.21
3	1.1	4.0	1.22
4	1.2	5.14	1.5
5	1.2	4.5	1.31
6	1.3	4.0	1.12
7	1.3	4.8	1.34
8	1.4	5.14	1.39
9	1.5	5.14	1.34
10	1.4	6.0	1.62

TABLE 4.1 Relationship between breaker height and initial run-up velocity for 10 selected waves.
Data recorded at Werri Beach - NSW.

measurements reported here are from the base of the slope rather than the mid swash-zone.

The breaker heights shown in table 4.1 can be related to the observed velocities at the base of the slope ($U_o(\text{obs})$) by:

$$U_o(\text{obs}) = K\sqrt{gH_b} \quad \dots(4.1)$$

where H_b is the observed breaker height. Values of K are shown in table 4.2 for the 10 waves analysed and fall between 1.12 and 1.62 with a mean of 1.36. It is interesting to compare these, first to equation 2.6 which predicts a velocity U_o in excess of $2\sqrt{gh}$ and second, to the example of a small bore collapsing on a flat beach (chapter 3) where this is found to occur. The measured breaker height is probably an over estimate of the bore height at the shoreline. However, the fact that all velocities at the base of the beach are substantially less than $2\sqrt{gH_b}$ suggests that equation 2.6 is inappropriate for determining U_o on a steep beach where there is clearly a great amount of energy lost in the highly turbulent breaking process.

Kirk (1975) and Waddell (1973) have published swash velocity data which can be compared to that presented here. Based on many measurements of individual swash flows from a steep gravel beach, Kirk derives a value equivalent to $K=1.28$, which is close to the mean of the 'K values' listed in table 4.1.

Waddell reports initial swash velocities which exceed \sqrt{gH} by a factor greater than 3. However, these were for

small breakers ($<0.4\text{m}$) in which the energy loss due to turbulence may not have been great.

4.3.2 SWASH EXCURSION WIDTH

Table 4.2 compares theoretical swash widths computed using equation 2.11 with observed widths. The results are shown graphically in figure 4.8 (a & b) with the vertical arrow representing the theoretical swash limit. Also shown is the position of the boundary between fully and partially saturated beach.

Very good agreement is seen between the frictionless theoretical solution and actual run-up width for small swash flows associated with low breakers. In these cases the swash lens never penetrates beyond the saturated beach. However, as the breaker height and swash width increase, the agreement breaks down. Swashes with a theoretical limit significantly beyond the intersection of the water table with the sand surface never reach this limit, which is indicative of the increase of percolation of water into the beach and of friction over the dry surface.

4.4 SUMMARY

Observations of breakers, bores and run-up on steep beaches show that:

(i) Major bores can be observed on steep beaches and are associated with plunging breakers. Minor bores also exist and are associated with collapsing and surging breakers. The conclusion is that an approach based on bore theory may prove fruitful when applied to the study of swash on

OBSERVATION NUMBER	LEADING EDGE VELOCITY AT END OF BORE COLLAPSE U_0 (m/sec)	THEORETICAL RUN-UP VEL. (m/sec)	OBSERVED RUN-UP VEL. (m/sec)
1	4.5	6.5	6.33
2	3.8	4.7	5.2
3	4.0	5.2	5.0
4	5.14	8.5	6.5
5	3.8	4.7	5.2
6	4.0	5.2	5.5
7	4.8	7.4	7.8
8	5.14	8.5	7.9
9	5.14	8.5	7.4
10	6.0	11.6	10.0

TABLE 4.2 Comparison of theoretical and observed run-up widths for 10 selected waves - Warri Beach, NSW.

Theoretical run-up widths are computed using equation 2.11, ie:

$$f_s = \frac{U_0^2}{2g \tan \alpha}$$

where f_s is the run-up width, U_0 is the velocity of the leading edge of water at the end of the bore collapse phase and $\tan \alpha$ is the beach slope, in this case, 0.158.

steep beaches.

(ii) The behaviour of major bores at the shoreline is in qualitative agreement with the theory. Collapse takes place over a width of 3-5 metres and water is propelled up the slope with a high initial velocity. This velocity however, is always lower than that given by equation 2.6. The high turbulence associated with breaking is suggested as a reason for this.

(iii) Actual swash excursion widths for cases where the lens of water does not travel beyond the saturated section of the beach correlate closely with theoretical widths predicted by the model in which gravity is the only decelerating force acting on the swash mass (eq. 2.11). This indicates that loss of energy due to friction may be insignificant over the lower beach face where swash velocities are high and the lens is travelling over a wet surface. However, the higher flows, which all penetrate onto the non saturated part of the beach, stop short of the theoretical limit, indicating that friction with the dry bed and percolation of water into the beach are important in determining maximum run-up widths.

The literature contains several reports concerning the relationship between inshore slope and the frequency of water motion in the inner surf zone and on the beach face. In general, it has been noticed that (i) wave frequencies on the extreme landward section of a gentle profile are significantly lower than incident wave frequency, and (ii) the frequency of swash decreases as beach slope decreases (eg. Emery and Gale, 1951; Huntley and Bowen, 1975). There are several explanations for the above but to date, no attempt has been made to model the processes which some have suggested may be important.

One such process is the merging of bores in the surf zone, the simulation of which can be achieved by using the theory of bore motion on a sloping beach (see Chapter 2). In this chapter, a model based on the theory is described and tested with a number of initial simplifying assumptions. The results obtained are discussed with reference to the field observations of the writer and others.

5.1 FIELD OBSERVATIONS OF SWASH PERIODS

The effect of beach slope on the frequency of swash

has been studied in the field by Emery and Gale (1951), Sonu et al. (1974), Huntley and Bowen (1975) and Bradshaw (1980). It is generally agreed that the period of the swash increases as the slope decreases and the most common explanation for this has to do with interaction between successive waves in the surf zone.

Emery and Gale (1951) and Sonu et al. (1974) suggest that multiple bores in the surf zone cause an impounding of water on the beach face which periodically gives rise to strong backwash. This may temporarily suspend the swash from several oncoming waves and lead to the apparent down shifting of wave frequency on the inner part of the profile. Huntley and Bowen (1975) argue against the importance of backwash interaction and explain the reduction in wave period across the surf zone in terms only of merging waves. Similar results from laboratory experiments have been published by Webber and Bullock (1968) who note that the merging of waves across the surf zone (slope 0.1) results in 30% fewer run-up crests than waves.

A study of the frequencies of swash and inner surf zone water motions on different types of beaches has been reported by Bradshaw (1980) and wave and current spectra from a steep beach and two flat beaches are reproduced in figures 5.1 to 5.3.

On the steep beach, the period of swash is seen to be very close to the that of the incident waves (figure 5.1). The data were collected under low energy conditions (breaker height < 0.5 metres) when most waves surged up the beach face for a short distance and returned completely before the arrival of the next wave. Huntley

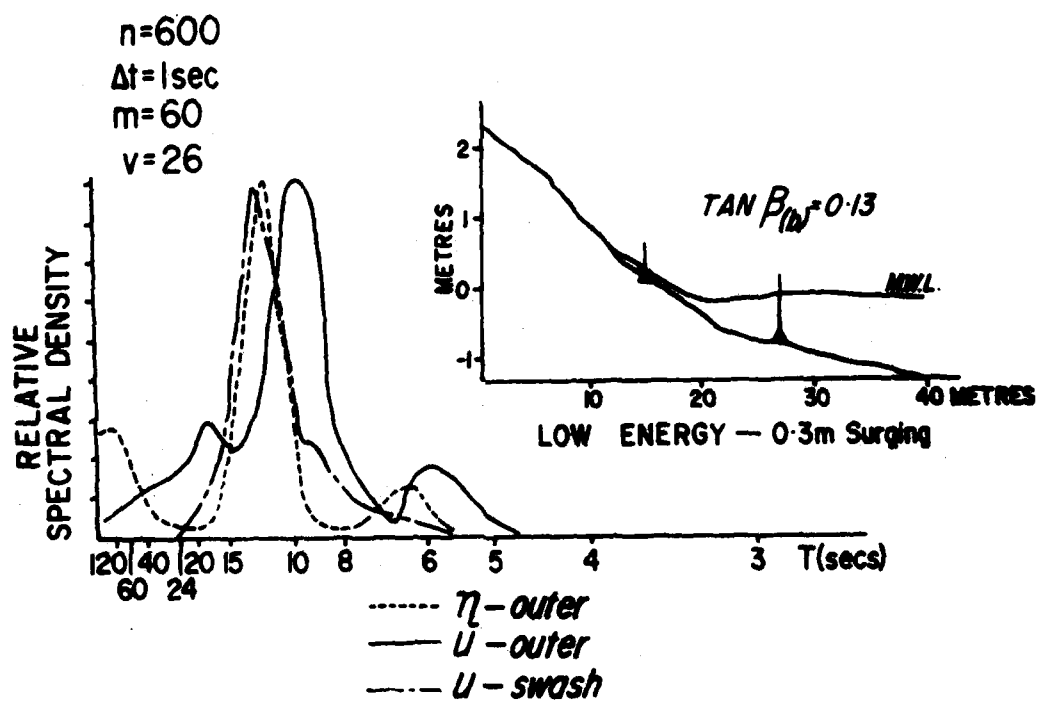


FIGURE 5.1 Run-up spectra from a low energy steep beach.
 (North Avalon, NSW) (from Bradshaw, 1980)

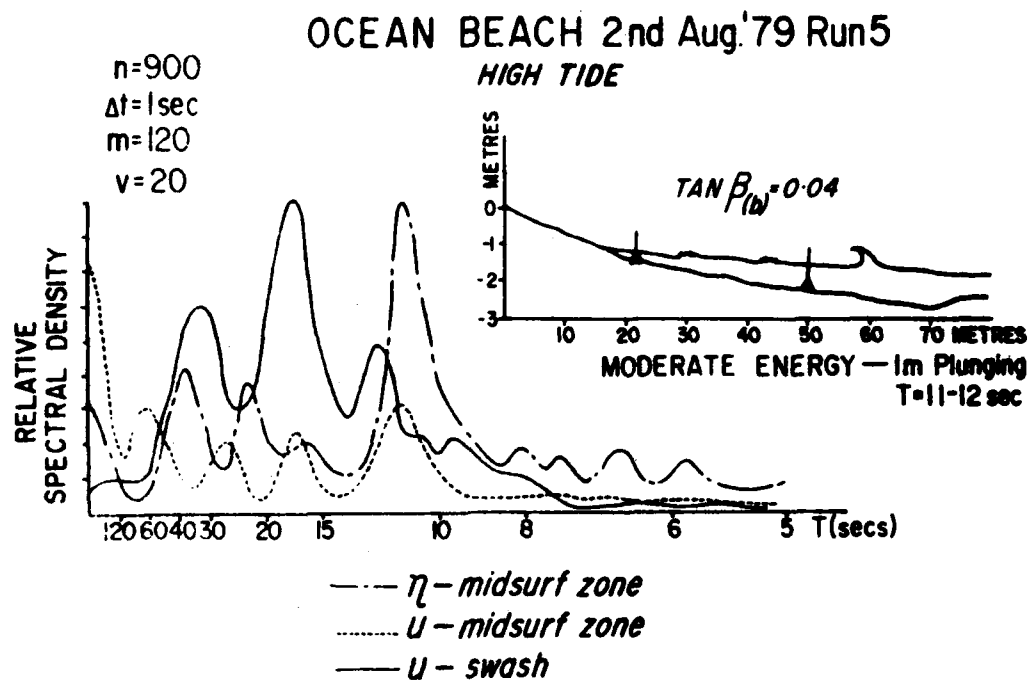


FIGURE 5.2 Surf zone and swash spectra from a low energy flat beach. (from Bradshaw, 1980)

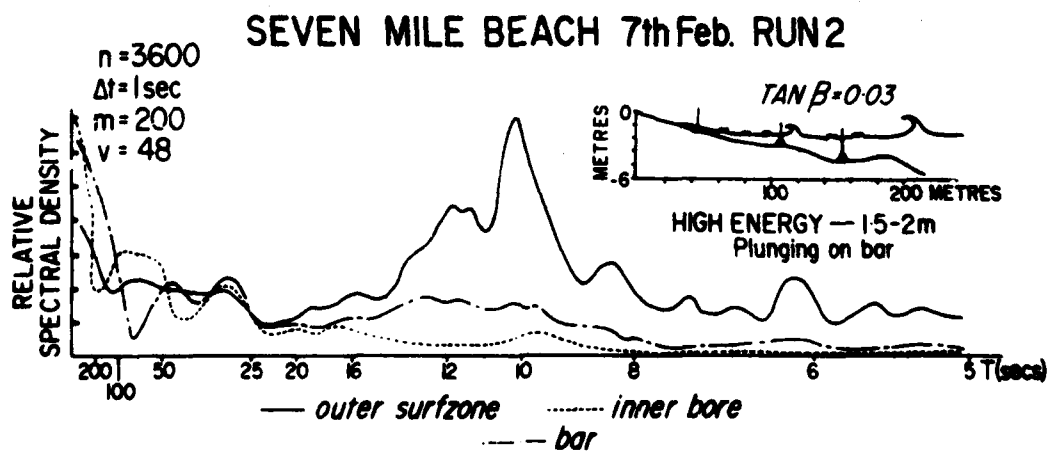


FIGURE 5.3 Surf zone and swash spectra from a high energy flat beach. (from Bradshaw, 1980)

and Bowen (1975) note that swash-backwash collision at the base of a steep beach (which usually seems to increase in intensity as energy rises) can often result in alternating high and low swashes but it is only occasionally that run-up from a shore break on a steep beach is totally suppressed by backwash from an earlier wave.

Figure 5.2 shows spectra from a low energy (≈ 1 metre breaker) flat profile (2.3°) where a single line of plunging breakers generated bores which often traversed the width of the surf zone without interference from following waves. It was often possible to relate swashes to individual breakers and the spectrum of water flow at the inner station reflects this, with a peak close to incident wave period. However, energy is dominated by peaks at lower frequencies with one obvious explanation being that not all bores generated at the break point succeeded in reaching the shoreline.

Figure 5.3 shows spectra from a higher energy, slightly flatter profile (1.5 - 2 metre waves on a 1.7° slope). Here, water motion in the inner surf zone is totally dominated by low frequencies with no energy at or near the frequency of incident waves. On this beach it was impossible to visually relate individual breakers to bores in the surf zone or to movement at the shoreline.

It seems clear from field observations that two related mechanisms can lead to the dominance of low frequencies in the swash on flat beaches. The first is the merging of successive waves in the surf zone and the second is the suppression of swash by strong backwash, caused by the accumulation of a large volume of water on the upper part of the profile. The rest of this chapter

deals with a procedure for modelling the first of these using only the theory of bore motion over a sloping bottom. Despite the initial simplifying assumptions, the model produces results which bear close resemblance to observations from nature and provides some theoretical insight into the behaviour of multiple bores on slopes with different gradients and incident wave energy conditions.

It is noted that other processes such as slow oscillations in the mean inshore water level (ie. surf beat) may contribute to low frequency energy at the shoreline (Holman, 1981; Huntley et al., 1981). However, these are not considered here.

5.2 DESCRIPTION OF THE MODEL

The model examines the extent to which bores are theoretically capable of merging across a surf zone, between the breaker line and the intersection of still water level with the slope. Run-up beyond still water level is not considered. Independent variables in the model are slope angle, wave energy (breaker height) and wave period and the merging process can be examined for different combinations of these. Bores are generated by a monochromatic wave train and progress into still or shoreward moving water; the effect of return flows are not considered.

Figure 5.4 is a definition sketch which shows a single bore moving across a surf zone, bottom slope α . For simplicity in the first instance, generating waves are assumed to have a shape shown by the shaded area and a

period, T . The bore starts at depth $d = 1.25H$ where H is the height of the wave. The initial steep face develops at time $t=0$, at position x_1 on the profile.

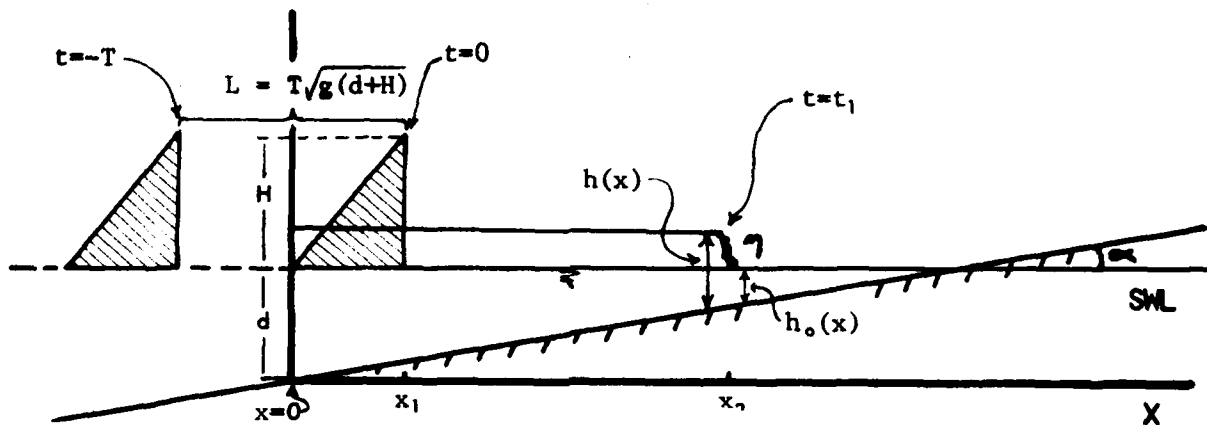


FIGURE 5.4 Definition sketch for single bore model.

The volume of water in the generating wave (K) per unit crest width is estimated by:

$$K = 0.25LH \quad \dots (5.1)$$

where $L = T\sqrt{g(d+H)}$

(note that distance from the origin to x_1 is assumed equal to $0.5L$).

To satisfy continuity, it is assumed that the volume of water in a bore above still water level at any time after breaking is equal to the volume of water in the generating wave. Thus, at time t_1 , the bore shown in figure 5.4 will have a volume per unit width given by:

$$\eta x_2 = 0.25LH = K \quad \dots(5.2)$$

where η is the height of the bore face.

At any point on the slope, the height of the bore face is given by:

$$\eta(x) = K/x \quad \dots(5.3)$$

The depth of water in front of and behind the bore face ($h_o(x)$ and $h(x)$ respectively) at any position x are given by:

$$h_o(x) = d - x \tan \alpha \quad \dots(5.4)$$

$$\text{and } h(x) = \eta(x) + h_o(x) \quad \dots(5.5)$$

Substituting equation 5.3 into the above gives:

$$h(x) = K/x + h_o(x) \quad \dots(5.6)$$

The velocity of the bore front at any position x can be found using equation 2.3 and substituting equation 5.6, ie:

$$\begin{aligned} dx/dt &= W(x) \\ &= \sqrt{\frac{g [K/x + h_o(x)] [K/x + 2h_o(x)]}{2h_o(x)}} \quad \dots(5.7) \end{aligned}$$

Therefore, the distance travelled by the bore over time Δt will be:

$$\Delta x = \Delta t \cdot W(x) \quad \dots(5.8)$$

By solving the above for successive time steps, starting from $t=0$, an $x-t$ diagram showing the motion of the bore across the profile can be constructed. The solution of equation 5.8 is obtained numerically using the fourth order Runge-Kutta method.

To test the procedure, a single bore was moved across a slope and pertinent parameters were listed at each time step. The bore in $x-t$ space is shown in figure 5.5 and the change in parameters associated with the bore are shown in tables 5.1 and 5.2. The first table covers the life of the bore from inception to the shoreline, with time steps of 1 second. Table 5.2 spans the last few seconds, just prior to the bore's arrival at the shoreline and is computed for a $\Delta t = 0.1$ seconds.

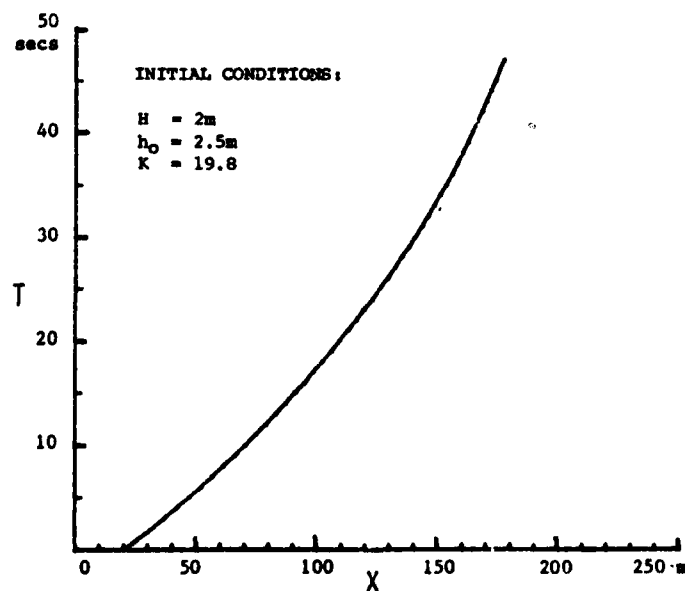


FIGURE 5.5 X-T diagram of a simulated bore.

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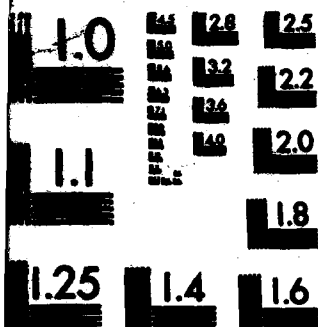
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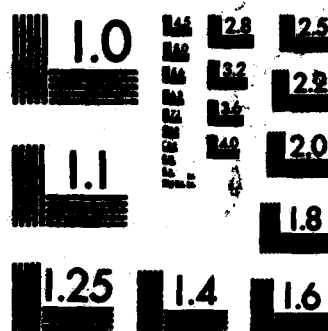
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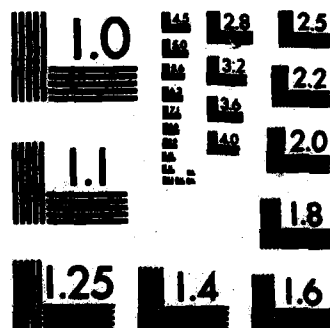
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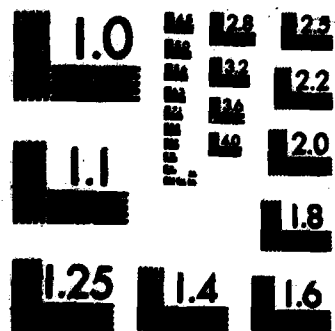
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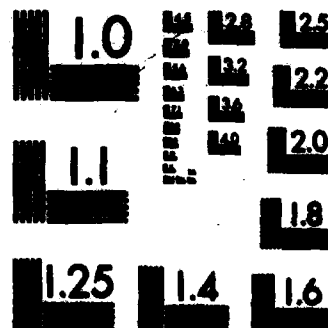
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t (secs)	x (m)	h_0 (m)	η (m)	γ	W (m/sec)	$w/\sqrt{gh_0}$
1.00	25.82	2.139	0.767	0.358	5.795	1.266
2.00	31.49	2.060	0.629	0.305	5.511	1.227
3.00	36.91	1.985	0.536	0.270	5.296	1.201
4.00	42.13	1.912	0.470	0.246	5.120	1.183
5.00	47.18	1.841	0.420	0.228	4.968	1.170
6.00	52.09	1.773	0.380	0.214	4.833	1.160
7.00	56.87	1.706	0.348	0.204	4.710	1.152
8.00	61.53	1.641	0.322	0.196	4.595	1.146
9.00	66.08	1.577	0.300	0.190	4.488	1.141
10.00	70.53	1.515	0.281	0.185	4.385	1.138
11.00	74.87	1.455	0.264	0.182	4.287	1.135
12.00	79.12	1.395	0.250	0.179	4.192	1.134
13.00	83.27	1.337	0.238	0.178	4.100	1.132
14.00	87.34	1.280	0.227	0.177	4.010	1.132
15.00	91.31	1.225	0.217	0.177	3.922	1.132
16.00	95.19	1.171	0.208	0.178	3.836	1.132
17.00	98.99	1.118	0.200	0.179	3.751	1.133
18.00	102.71	1.066	0.193	0.181	3.667	1.135
19.00	106.34	1.015	0.186	0.183	3.585	1.137
20.00	109.89	0.965	0.180	0.187	3.504	1.139
21.00	113.36	0.917	0.175	0.190	3.423	1.142
22.00	116.75	0.870	0.170	0.195	3.343	1.145
23.00	120.06	0.823	0.165	0.200	3.264	1.149
24.00	123.30	0.778	0.161	0.206	3.186	1.154
25.00	126.45	0.734	0.157	0.213	3.108	1.159
26.00	129.53	0.691	0.153	0.221	3.031	1.165
27.00	132.53	0.649	0.149	0.230	2.955	1.171
28.00	135.45	0.609	0.146	0.240	2.878	1.179
29.00	138.30	0.569	0.143	0.252	2.803	1.187
30.00	141.07	0.530	0.140	0.265	2.728	1.197
31.00	143.76	0.493	0.138	0.280	2.653	1.208
32.00	146.39	0.456	0.135	0.297	2.579	1.220
33.00	148.94	0.420	0.133	0.316	2.506	1.235
34.00	151.41	0.386	0.131	0.339	2.433	1.251
35.00	153.81	0.352	0.129	0.365	2.361	1.271
36.00	156.15	0.320	0.127	0.397	2.290	1.294
37.00	158.41	0.288	0.125	0.434	2.220	1.321
38.00	160.60	0.257	0.123	0.479	2.151	1.354
39.00	162.72	0.228	0.122	0.534	2.083	1.394
40.00	164.78	0.199	0.120	0.603	2.018	1.445
41.00	166.77	0.171	0.119	0.693	1.956	1.510
42.00	168.70	0.144	0.117	0.813	1.899	1.597
43.00	170.58	0.118	0.116	0.983	1.850	1.719
44.00	172.41	0.093	0.115	1.241	1.815	1.906
45.00	174.22	0.067	0.114	1.689	1.808	2.227
46.00	176.04	0.042	0.112	2.690	1.883	2.942
47.00	178.05	0.014	0.111	8.028	2.479	6.728

TABLE 5.1 Properties of simulated bore from inception to the vicinity of the shoreline.

Initial Conditions: $H = 2m$
 $h_0 = 2.5m$
 $K = 19.8$

t (secs)	x (m)	h ₀ (m)	η (m)	γ	w (m/sec)	w $\sqrt{gh_0}$
45.00	172.43	0.092	0.115	1.244	1.815	1.908
45.10	172.61	0.090	0.115	1.277	1.812	1.932
45.20	172.79	0.087	0.115	1.313	1.810	1.957
45.30	172.97	0.085	0.114	1.351	1.809	1.984
45.40	173.15	0.082	0.114	1.391	1.807	2.013
45.50	173.33	0.080	0.114	1.433	1.806	2.044
45.60	173.51	0.077	0.114	1.478	1.806	2.076
45.70	173.69	0.075	0.114	1.527	1.806	2.111
45.80	173.87	0.072	0.114	1.579	1.806	2.148
45.90	174.05	0.070	0.114	1.634	1.807	2.188
46.00	174.23	0.067	0.114	1.694	1.809	2.230
46.10	174.41	0.065	0.114	1.758	1.811	2.277
46.20	174.60	0.062	0.113	1.828	1.814	2.327
46.30	174.78	0.059	0.113	1.904	1.818	2.381
46.40	174.96	0.057	0.113	1.987	1.823	2.440
46.50	175.14	0.054	0.113	2.078	1.829	2.505
46.60	175.32	0.052	0.113	2.178	1.837	2.577
46.70	175.51	0.049	0.113	2.289	1.846	2.656
46.80	175.69	0.047	0.113	2.413	1.857	2.744
46.90	175.88	0.044	0.113	2.553	1.870	2.844
47.00	176.07	0.041	0.112	2.711	1.885	2.957
47.10	176.26	0.039	0.112	2.893	1.904	3.086
47.20	176.45	0.036	0.112	3.103	1.926	3.235
47.30	176.64	0.033	0.112	3.350	1.953	3.411
47.40	176.84	0.031	0.112	3.645	1.987	3.621
47.50	177.04	0.028	0.112	4.006	2.028	3.877
47.60	177.24	0.025	0.112	4.458	2.080	4.198
47.70	177.45	0.022	0.112	5.045	2.148	4.615
47.80	177.67	0.019	0.111	5.845	2.240	5.182
47.90	177.90	0.016	0.111	7.013	2.370	6.009
48.00	178.15	0.012	0.111	8.923	2.572	7.362
48.10	178.42	0.009	0.111	12.781	2.944	10.092
48.20	178.74	0.004	0.111	26.760	4.024	19.980

TABLE 5.2 Properties of a simulated bore in the vicinity of the shoreline.

The following changes in properties of the bore are noted from tables 5.1 and 5.2. and are found to be in agreement with similar numerical results published by Keller et al. (1960):

- (i) Bore height decreases as $h_o \rightarrow 0$,
- (ii) Bore height-to-depth ratio (δ) decreases for a short distance but then increases all the way to the shoreline,
- (iii) Bore velocity (W) decreases until $W/\sqrt{gh_o} \approx 2.11$, after which it increases to the shoreline. (Note: Keller et al., 1960, find that minimum bore velocity occurs at $W/\sqrt{gh_o} = 2.504$),
- (iv) Near the shoreline there is a very rapid increase in both δ and W .

Now the model is extended to deal with multiple bores.

With reference to figure 5.6, it is assumed that the flows of bore 1 and bore 2 are independent and that K is the same for both bores.

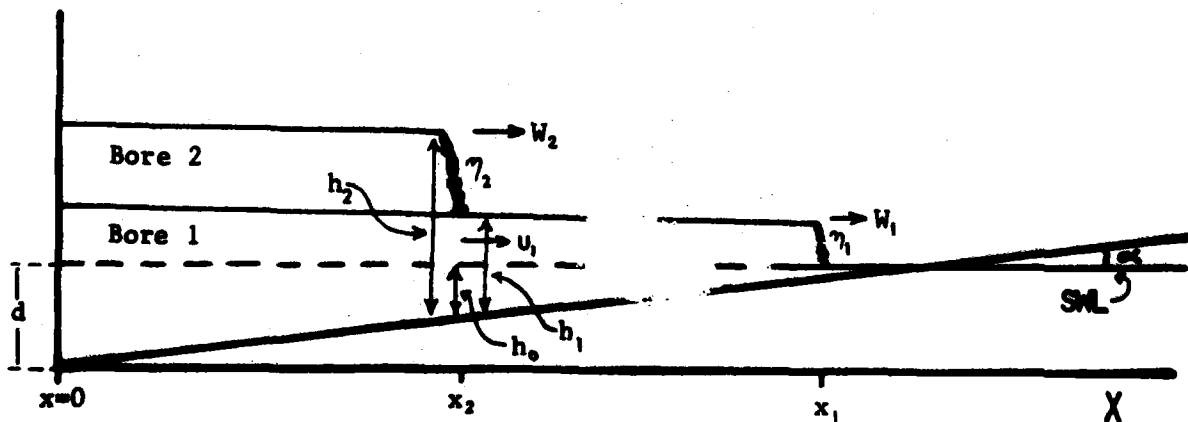


FIGURE 5.6 Definition sketch for multiple bore model.

Bore 1 moves into still water and is modelled in the manner discussed above. Bore 2 moves into water which has a positive (shoreward) velocity, u . To obtain an estimate of u at the position of the second bore face we assume that water particle velocity behind the bore face decreases linearly with distance. Then, u_1 at position x_2 (in figure 5.6) is:

$$u_1 = W_1 \cdot (x_2/x_1) \quad \dots (5.9)$$

The water depths in front of and behind the second bore face (h_1 and h_2 respectively) will be:

$$h_1(x_2) = d - x_2 \tan \alpha + \eta_1 \quad \dots (5.10)$$

$$\begin{aligned} \text{and } h_2(x_2) &= h_1(x_2) + \eta_2 \\ &= h_1(x_2) + K/x_2 \end{aligned} \quad \dots (5.11)$$

Then, from equation 2.3,

$$\begin{aligned} W(x_2) &= dx_2/dt \\ &= \sqrt{\frac{g[K/x_2 + h_1(x_2)][K/x_2 + 2h_1(x_2)]}{2h_1(x_2)}} + u_1 \end{aligned} \quad \dots (5.12)$$

$$\text{and } \Delta x_2 = \Delta t \cdot W(x_2) \quad \dots (5.13)$$

The numerical solution of equations 5.12 and 5.13 yields an x - t diagram for the second bore.

When the first bore is overtaken by the second at some position x , then a new bore is formed with a height η_3 , given by:

$$\begin{aligned}\eta_3(x) &= \eta_1(x) + \eta_2(x) \\ &= 2K/x\end{aligned}\quad \dots(5.14)$$

The initial model has been constructed to produce up to four bores which begin in succession with a constant time lag T , where T is the breaker period. All bores produced have the same initial height and the simulation runs until a bore front reaches the shoreline. A time step of 0.5 seconds is used throughout.

5.3 RESULTS AND DISCUSSION

The simulation has been carried out for three different combinations of the independent variables:

- (i) Slope angle is varied from 0.5° to 2° while holding breaker height and period constant at 2 metres and 8 seconds respectively (figure 5.7)
- (ii) Breaker height is varied from 0.5m to 3m. Slope and period held constant at 1° and 8 seconds respectively (figure 5.8)
- (iii) Wave period is varied from 6 to 12 seconds with a constant slope (0.8°) and wave height (2 metres) (figure 5.9).

Figure 5.7 is most important in terms of previous observations from the field. It demonstrates that:

- (i) It is theoretically possible for bores to merge across a surf zone in the manner described by Huntley and Bowen (1975) and Webber and Bullock (1968) even when these are generated by waves of constant period and height. The

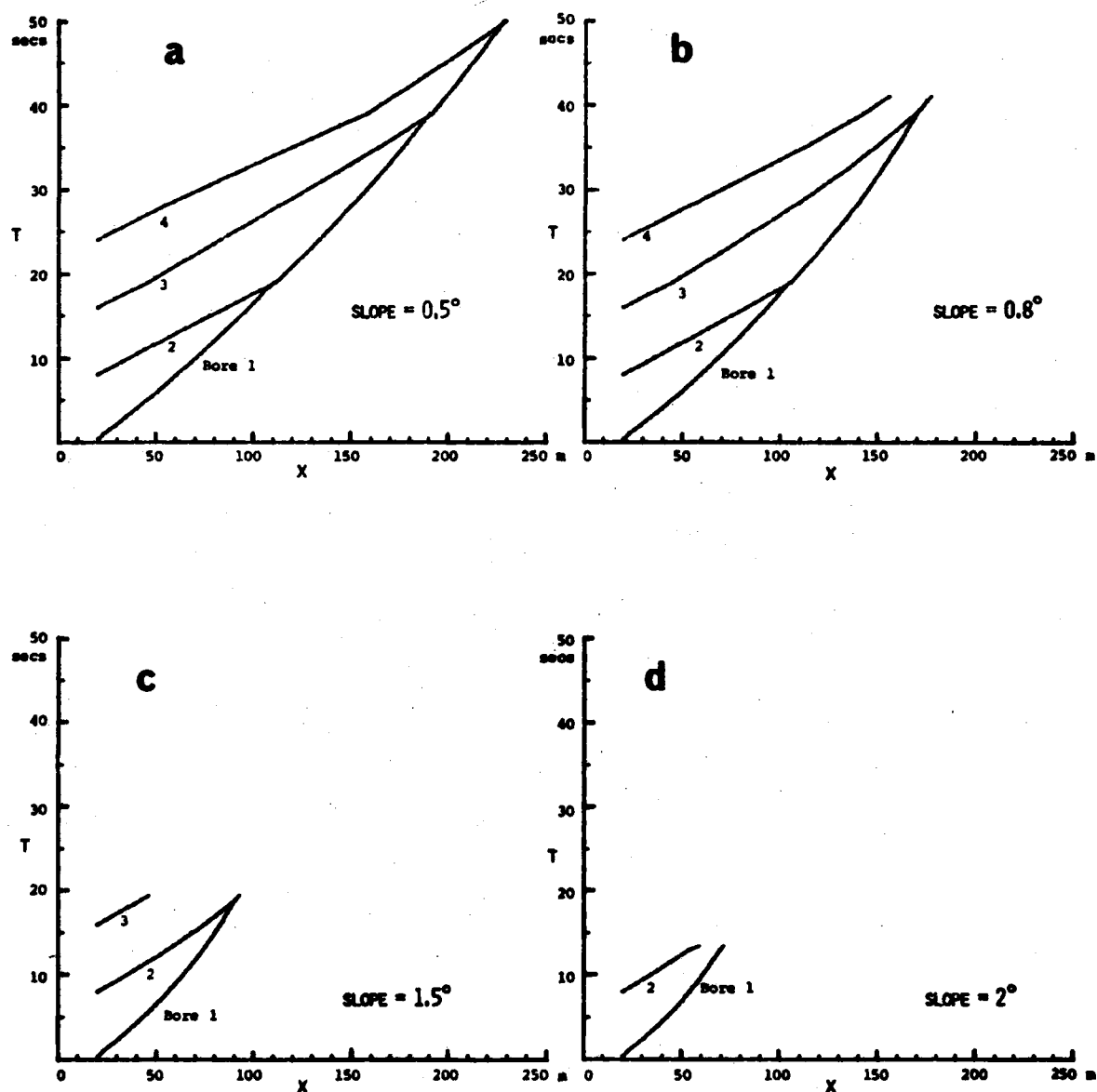


FIGURE 5.7 Behaviour of multiple bores on four different slopes with constant wave height (2m) and wave period (8 seconds).

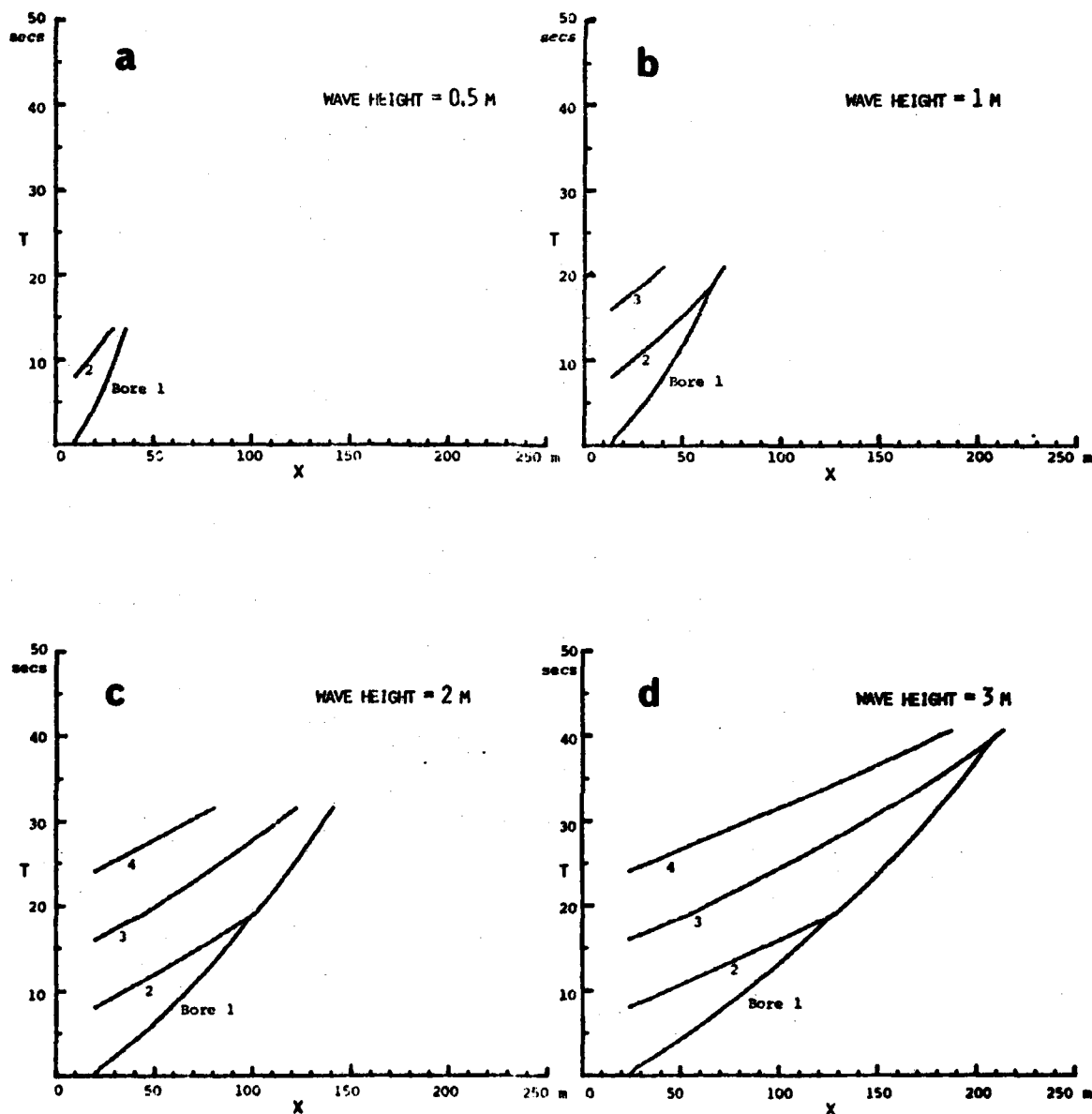


FIGURE 5.8 Behaviour of multiple bores on a 1° slope with varying wave energy levels. Wave period held constant at 8 seconds.

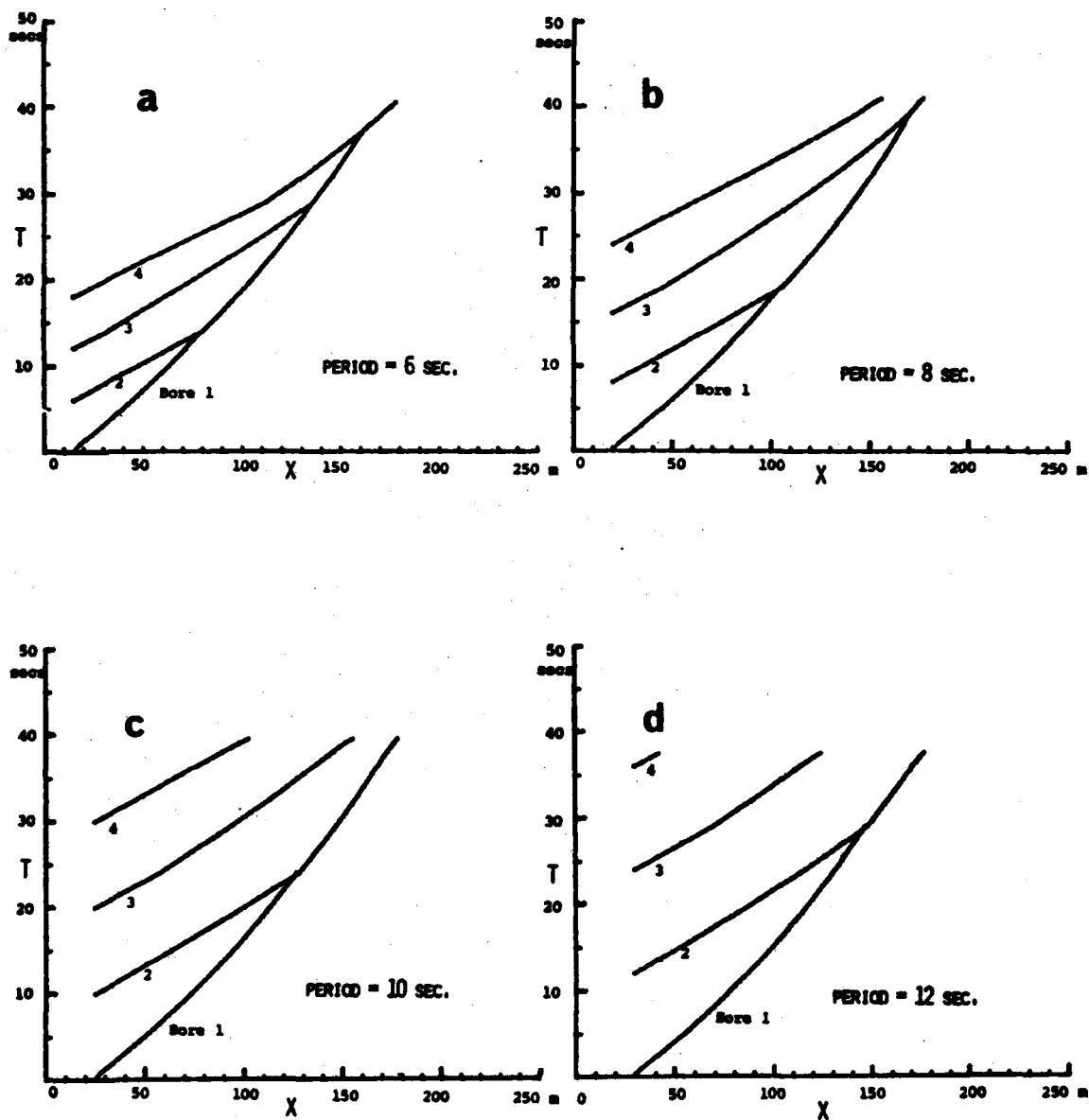


FIGURE 5.9 Behaviour of multiple bores on a 0.8° slope with varying wave period. Wave height held constant at 2m.

analysis shows that sets of high and low breakers are not a prerequisite for bore-bore capture.

(ii) The model predicts the relationship between inshore slope and swash frequency that is found in nature. Figure 5.7 shows that as slope decreases, the surf zone widens, more bores merge and periods at the shoreline will increase relative to incident wave period.

The general nature of expected shoreline frequencies can be inferred from figure 5.7. On a 2° slope, one would expect to see some incident wave energy at the beach face because bores are able to travel the width of the surf zone without being overtaken. However, on the more gentle slope of 0.5° , over which four bores are capable of merging, dominant periods at the shoreline should be in excess of $4T$.

Observations shown in figures 5.2 and 5.3 are in agreement with this inference. Although the 2.3° slope (figure 5.2) shows a reduction in incident frequency and a growth of lower frequencies in the swash zone relative to the mid surf zone, most energy in the swash lies at periods less than $2T$. This contrasts to the flatter 1.7° slope where there is a total absence of energy in the inner surf zone in the range T to $2T$.

Figures 5.8 and 5.9 show the theoretical relationship between breaker height and breaker period on the one hand, and propensity for bores to merge on the other. As breaker height increases, the model predicts a widening of the surf zone and an increase in the number of bores that merge. This may partly explain observations that low frequencies on the beach grow as incident wave energy increases (Guza and Thornton, 1982). It may also be an

important consideration in interpreting data shown in figures 5.2 and 5.3 since these two flat beaches represent different energy conditions as well as different slopes. The total dominance of low frequencies evident in figure 5.3 may be the result of a combination of both the moderately high waves (1.5m - 2m) and the flat profile.

Figure 5.9 gives the expected result that, as the interval between breakers decreases, there is an increase in the number of bores which exist simultaneously in the surf zone, and which combine over its width. It can be inferred that the relative down-shift in frequency across the surf zone will be greatest when incident wave frequency is high.

The combined effects of slope and incident wave frequency have been expressed by many (eg. Guza and Inman, 1975) using the dimensionless parameter, ϵ , given by:

$$\epsilon = a\omega^2/g\tan^2\alpha \quad \dots(5.15)$$

where a is incident wave amplitude,

and ω is incident wave radial frequency.

The analysis suggests that increasing values of ϵ will correspond to more pronounced frequency down-shifting on the beach relative to incident wave frequency.

The analysis in figures 5.7-5.9 directly addresses the question of whether bores are theoretically able to merge across a surf zone and bring about a decrease in dominant frequency as the shoreline is approached. However, the related process of bore-backwash interaction, which Emery and Gale (1951) and Sonu et al. (1974) consider as being most important, is not dealt with.

Nevertheless, some inferences can be made from the analysis regarding the effects of backwash and here I use figure 5.8c as a convenient example. Starting at $t=0$ with a still surf zone, the graph indicated that the first two bores will merge before the shoreline is reached and it seems likely that an extension of the analysis to include run-up would show the third bore overtaking or at least contributing to the swash generated by the combination of bores 1 and 2. It also seems likely from the the diagram that the fourth bore will fail to arrive at the shoreline before backwash from the previous three has begun. Given that the backwash, which is the combined volume of water of 3 bores, will be large and fast flowing, it is plausible that the fourth and possibly subsequent bores will be suppressed to the extent that they produce little or no run-up. This is the process described by Sonu et al. (1974) and also noted in chapter 3 of this report. The periodic occurrence of powerful backwash which inhibits further swashes for a considerable length of time can be readily observed on most flat beach.

The conclusion is that two bore and swash related processes are capable of contributing to low frequencies observed on the inner sections of flat beaches. In the first instance frequency down-shifting will occur because bores overtake their predecessors and this will further be enhanced by the suppression of some bores due to the periodic occurrence of strong backwash.

It should be stressed here that these processes may not account fully for all low frequencies observed. Several (eg. Van Dorn, 1976; Guza and Thornton, 1982; Huntley et al., 1977) correctly argue that the total

shoreline movement must be viewed as a combination of a super-elevation of surf zone water level (set-up) on which higher frequency waves are superimposed. If mean inshore water level fluctuates at surf beat frequencies due, say, to periodic changes in the characteristics of incident wave groups, then this will register at the shoreline. However, the analysis does clearly show that it is possible to explain the low frequencies in swash by a simple consideration of the theory of bore motion over a sloping bottom, without recourse to any arguments concerning external low frequency forcing.

The foregoing discussion, particularly concerning the role of backwash, highlights areas where refinement and additions to the model are necessary. In the first instance, the simulation should be extended to include swash flows over the zone of temporary inundation at the top of the beach. This will give a complete picture of the degree to which waves are capable of merging over the entire inshore zone from the break point to the point of maximum run-up. This should not prove difficult since the theory necessary for the addition is well documented (Freeman and LeMehaute, 1964), at least for the friction-free case. The degree to which friction influences the flow of a thin sheet of water over a gently sloping sandy bed requires a great deal of investigation.

The inclusion of backwash in the model presents more formidable problems since it requires a knowledge of the mechanics of the interaction between a high velocity seaward flowing sheet of water and an incoming bore. While data presented in chapter 3 indicate that return flows can be dealt with in the inner surf zone (where seaward velocities are usually less than 1 m/sec - see table 3.1),

this may not be the case further landward where water depths are very small, return flow velocities high, and where highly turbulent collisions are often observed.

On a more practical level, further refinement of the model could be made by finding more realistic approximations for breaker volume and the change in water depth and particle velocity with distance seaward of the bore crest.

6.1 BORES

Bores are a common feature of natural surf zones and have been the subject of substantial theoretical work. Surprisingly, they have not been investigated in the field, and on this basis some experiments were carried out to provide initial data in this area. These show that the theory qualitatively predicts some of the changes in bore properties that are observed as a bore approaches the shoreline. In particular, the theory predicts an increase in the bore height-to-water depth ratio as the bore progresses into very shallow water and this seems to occur in nature. Of far greater significance is the fact that observed bore velocities, measured over a range of depths (0.04-0.4m) and for cases of bores moving into both still and seaward flowing water, correlate closely with those predicted by the theory. The small theoretical over-prediction found in each case may be due to (i) experimental error in the measurement of water velocities and water depths, and/or to (ii) the fact that the theory does not consider loss of energy resulting from turbulence in the steep face and from bottom friction.

Turbulence and bottom friction are obviously important

in the consideration of any wave motion in the surf zone, but few field measurements have been made to quantify these. A great deal of theoretical and field based study is required to provide the information needed to refine wave models such as the one considered here. However, as a first approximation, the existing theory of bore motion over a sloping bottom seems to be adequate for describing bores on a natural beach.

The recording techniques used introduced some degree of experimental error, associated mainly with the estimation of return flow velocities. However, cine-photography proved to be a most effective way of gathering high quality data on wave shapes, crest velocities and relative water depths, and it's future use for surf zone work is strongly recommended, especially in conjunction with electronic measurement of flow velocities.

An important point arising from the investigation is that there are, at least theoretically, fundamental differences in the properties of waves which can coexist in a surf zone, and this must be taken into account in future field studies. Bores in the inner surf zone must be treated differently to the non-saturated breakers which seem to occur in deeper water, especially if serious modelling of surf zone processes is to be attempted.

To fully understand the nature of surf zone waves it is essential that some initial work be carried out to identify the spatial domains of different wave types and the conditions (especially topographical) under which waves change from one type to another. So far, this has only been outlined at a theoretical level by LeMehaute (1962).

On the basis of the results obtained from a comparison of theoretical and actual bore properties, the theory was used to model the behaviour of multiple bores in a surf zone. The existence of several bores in a surf zone at any one time is readily observed in nature and has been suggested as a cause of low frequencies evident in the swash on flat profiles. The model presented is based on a number of simplifying assumptions and is thus a prototype. Specifically, it models bore travel between the break point and the beach face only and does not consider merging in the swash zone or the effects of bore-backwash interaction. Nevertheless, the simulation shows that over-running bores can be predicted from a consideration of bore theory alone, and provides a basis for interpreting field observations which show a shift to low frequencies towards the landward side of wide, flat surf zones. Extension of the model to include swash and return flows will provide valuable insight into inner surf zone processes. However, this extension will not be possible until the mechanics of bore-backwash interaction have been documented at a detailed level.

6.2 SWASH

Bores in the inner surf zone have been linked to swash flows on the 'dry' beach by a model which predicts a collapse of the steep bore face at the shoreline, followed by an explosive surge of water up the beach face. The lens of water on the beach is assumed to decelerate under the influence of gravity only. Cine-films made on steep and flat beaches provide some preliminary information on the usefulness of this model and highlight areas for future work.

Analysis of the films reveals that:

(i) A flattening of the bore face does occur on both slope extremes. The bore-swash transition region varied in width from 1.5m (small bore, flat slope) to 5m (large bore, steep slope).

(ii) The leading edge of water experiences an acceleration during bore collapse which was greater, relative to bore height, for the small bore on the flat beach. This is probably due to the high turbulence and energy losses associated with wave breaking and bore development on steep slopes.

(iii) Swash flows on both types of beaches are not adequately predicted by a model which considers gravity only. This is particularly the case on a steep profile where the potential exists for significant percolation of water into the coarse sediment that is a characteristic of this beach type.

The preliminary observations indicate that future research should be carried out in two areas. First there is a need to identify factors which influence leading edge velocities associated with the disappearance of the bore front. This is ideally done using photographic techniques but will present problems where bore collapse occurs over a wide expanse of beach face (flat profile) or where the collapse is associated with a high degree of turbulence (steep profile). Second, the effects of friction on the thin swash lens and loss of water due to percolation must be quantified and related to beach slope. Incorporation of these into the existing model is needed if accurate descriptions of swash flows are to be obtained.

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